VOLUME 86 NO. SM1
FEBRUARY 1960
PART 1

JOURNAL of the

Soil Mechanics and Foundations Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles must have a length not exceeding 50 characters and spaces.

2. A 50-word summary must accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper. Specific reference and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-in. by 11-in. paper. Because illustrations will be reproduced with a width of between 3-in. and 4½-in., the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. Papers should average about 12,000 words in length and must be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

Reprints from this Journal may be made on condition that the full title of the paper, name of author, page reference, and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

Subject and author indexes, with abstracts, are published at the end of each year for the Proceedings of ASCE. The index for 1958 was published as Proc. Paper 1891; indexes for previous years are also available.

PO,SA,SM.

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

SOIL MECHANICS AND FOUNDATIONS DIVISION
EXECUTIVE COMMITTEE
Jorj O. Osterberg, Chairman; Thomas M. Leps, Vice Chairman;
Stanley J. Johnson; Reginald A. Barron; Gerald A. Leonards, Secretary

COMMITTEE ON PUBLICATIONS
H. Bolton Seed, Chairman; John A. Focht, Jr.; Harold J. Gibbs;
James P. Gould; T. William Lambe; Ralph B. Peck; Frank E. Richart, Jr.;
Woodland G. Shockley; Thomas H. Thornburn

CONTENTS

February, 1960

Papers

	Page
Dynamic Testing of Pavements by W. Heukelom and C. R. Foster	1
Anchored Bulkhead Design by Numerical Method by F. E. Richart	29
Installation and Operation of Dewatering Systems by David A. Werblin	47
Discussion	67

Copyright 1960 by the American Society of Civil Engineers.

Journal of the
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

DYNAMIC TESTING OF PAVEMENTS

By W. Heukelom[1] and C. R. Foster,[2] M. ASCE

SYNOPSIS

Each time a vehicle passes over a pavement the surface is deflected and rebounds, creating temporary strain conditions for 0.01 to 0.1 sec. Information about these strain conditions can be obtained by observing the movements of the pavement under similar dynamic loading conditions. Methods used for the dynamic investigation of pavements, base courses, and subgrades are described in the present paper.

INTRODUCTION

The investigation of pavements and paving materials aims at finding constructions for highways, runways, taxiways, parking areas and other surface areas which are capable of carrying traffic or parked vehicles without failure during a sufficiently long time. In these various applications the loading conditions show considerable differences.

L. W. Nijboer[3] classified the loading conditions as follows:

Stationary Vehicles.—A normal stress is exerted upon the pavement for relatively long times, varying between 1/4 hour and several hours. Indentations can be formed in asphalt and other bituminous pavements at high tem-

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SM 1, February, 1960.

[1] Koninklijke/Shell-Laboratorium, Amsterdam (N. V. De Bataafsche Petroleum Naatschappij).

[2] Ass't. Chf., Soils Div., U. S. Army Engr. Waterways Experiment Sta., CE, Vicksburg, Miss.

[3] "Strasse und Autobahn," by L. W. Nijboer, vol. 8, (1957), p. 210.

peratures. Plastic subgrades like clay and peat can show irreversible deformations. The stability of these materials under a standing load can be assessed, for example, by using triaxial shear methods at very low rates of deformation as described by W. Smith[4] and L. W. Nijboer.[5]

Braking and Accelerating Traffic.—The pavement is subjected to both normal and shearing stresses for moderate times, 1/4 to 1 second, which - at temperatures - can lead to rutting and waviness. The stability of the paving material must now be assessed by a "fast" test, such as the well-known Marshall test, at a suitable high temperature.[3]

Fast Moving Traffic.—Normal stresses are acting on the pavement for relatively short times varying between 1/100 and 1/10 of a second. As will be shown later, high bending stresses can occur in the pavement - in particular at low temperatures in the case of asphalt - which can cause cracks due to fatigue rupture in the paving material. This phenomenon can be investigated by means of dynamic testing methods.

As intimated in the preceding paragraphs, the mechanical properties of asphaltic paving materials are dependent upon the temperature and the loading time. Asphaltic bitumen may behave as an elastic material but also as a liquid, according as the loading time is very short or very long. Recent publications[6,7,8,9] have stressed the importance of this phenomenon.

These considerations prompted the development of methods for the investigation of asphalt throughout the range of temperatures and loading times which may occur in practice. Traffic being essentially dynamic, special attention has been paid to the investigation of the dynamic properties of asphalt and roads.

As cracks develop after many repeated loadings, it will be obvious that fatigue plays a part. An investigation programme was started, ten years ago, to find a non-destructive method for determining the stresses to which pavements are exposed under traffic circumstances and for comparing these stresses to the fatigue strength of the paving materials under similar dynamic circumstances. This programme was worked out by Nijboer, and it resulted in a method to compute the thickness which a pavement must have so that a given type of traffic cannot cause excessive stresses and strains in the surface.[10] The present paper gives a survey of the dynamic methods of investigation used for this purpose.

---

4 "Manual on Hot-mix Asphaltic Concrete Paving," by W. Smith, Construction Series No. 72, The Asphalt Institute, 1950.

5 "Plasticity as a Factor in the Design of Dense Bituminous Road Carpets," by L. W. Nijboer, Elsevier Publishing Co., Inc., Amsterdam, 1948.

6 "Rheology and applied mechanics," by R. N. J. Saal, and C. van der Poel, "Representation of rheological properties of bitumens over wide range of temperatures and loading times." Proceedings, Second International Congress on Rheology, Oxford (July, 1953), pp. 28 and 331.

7 "General system describing visco-elastic properties of bitumens and its relation to routine test data," by C. van der Poel, Journal of Applied Chemistry, vol. 4, part 5 (May 1954), p. 221.

8 Chapter IX of: "M. Reiner, Building Materials, Their Elasticity and Inelasticity," by C. van der Poel, Amsterdam, 1955.

9 "Time and temperature effects on deformation of asphaltic bitumens and bitumen-mineral mixtures," by C. van der Poel, Journal, Society Plastics Engineers, vol. 11, No. 7 (Sept 1955), p. 47.

10 "Rev Gén. Routes et Aérodromes," by L. W. Nijboer, vol. 27, No. 302 (1957), p. 63.

STRESS AND STRAIN IN A PAVEMENT UNDER TRAFFIC CONDITIONS

Measuring Technique.—Direct stress measurements in a pavement would require built-in dynamometers. As the measurements should have a non-destructuve character, this method cannot be applied. The alternative is to measure the strain ϵ and compute the stress σ as given by:

$$\sigma = \epsilon E_S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which E_S is the dynamic modulus of elasticity (Young's or E-modulus) of the pavement.

Non-destructive strain measurements can be carried out with the aid of strain gauges and electric recording equipment. Strains occurring in pave-

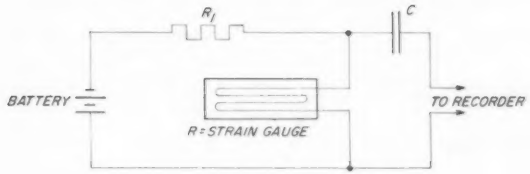


FIG. 1.—CIRCUIT FOR DYNAMIC STRAIN MEASUREMENTS

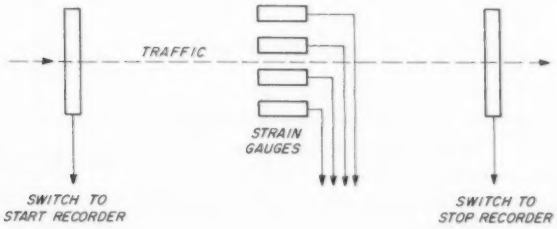


FIG. 2.—POSITION OF STRAIN GAUGES AND SWITCHES ON THE PAVEMENT

ments under moving traffic were measured with Philips strain gauges cemented to the surface by means of special Philips cement. This procedure was first checked in the laboratory on samples of asphalt mortar which were exposed to known vibrations, so that the strains could be computed and compared with the values measured. The experimental and calculated values agreed to within about 10%.

The wiring system used is shown in Fig. 1. A battery causes a current to run through the strain gauge and a constant resistance R_1. A change in the length of the material to which the strain gauge is cemented causes a change in the resistance of the strain gauge. The strain in the material induces a relative change in the resistance which is recorded as the wheel passes over the strain gauge. This is automatically done by the passing wheel, which starts and stops the recorder or the oscilloscope time base when it touches the switches shown in Fig. 2.

An a-c signal of known voltage is recorded from time to time providing a scale for the actual strains.

Results Obtained.--The records of strains occurring under moving traffic have a shape as shown in Fig. 3(a). A compression is found under the center of the tire while two equal extensions are found at longer distances from the center. In this case the maximum value of the extensions is equal to one-fifth of the maximum compression. This type of record can be explained by bending of the pavement as shown in Fig. 3(b) assuming that the neutral fiber is below the top of the construction.

In some cases the pavement may skid on the base course or a relatively large shear deformation may occur in the pavement. In these cases the record is distorted, so that it consists of a compression range and one extension

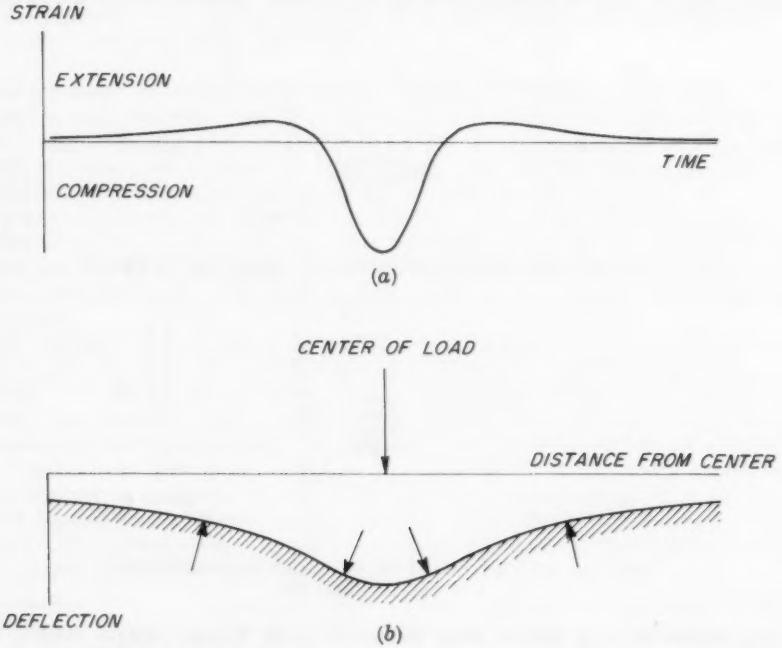


FIG. 3.—RECORDED STRAIN IN COMPARISON WITH DEFLECTION OF A PAVEMENT

range only, where the extension is usually greater than one-fifth of the compression. We shall consider only the symmetric record, as normally found.

If the same vehicle passes several times over the same strain gauge the observed strains show a scatter of about 30% around the average value which is not due to lateral placement of the wheel. An example of a series of measurements is given in Fig. 4, showing the integral distribution curve of the strains measured on an asphaltic concrete road with a wheel load of 2.8 tons at a speed of 25 mph. As this scatter must be attributed to bouncing of the wheels, it follows that the maximum shock coefficient is about 1.3 for this example.

The magnitude of the observed strains varies between 10^{-5} and 10^{-3} depending on the load and the strength of the construction.

Relations Between Load and Construction Data and Strain.—The observed strain can be compared to the strains produced in laboratory fatigue strength tests on the paving material as a possible means of predicting the probable lifetime of the pavement. The quality of existing road constructions can be judged in this way.

For design purposes, however, this method of investigation does not provide sufficient information, as can be demonstrated by considering the relation between the strain (or stress), and the load and the construction data.

The theoretical relationship between these variables has been treated by several authors. For our purpose, only A. H. A. Hogg's approach[11] will be discussed. Hogg calculated the bending stress generated at the bottom of a single elastic slab on homogeneous elastic subgrade if a static load is applied. The results have been prepared in a graphical form by N. Odemark[12] as

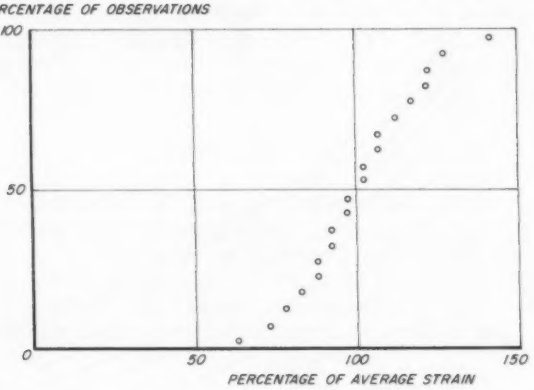


FIG. 4.—INTEGRATED DISTRIBUTION CURVE OF OBSERVED STRAINS

shown in Fig. 5, where the ratio

$$\frac{\sigma_b}{\sigma_o} = \frac{\text{bending stress at the bottom of the slab}}{\text{normal stress applied to the top}}$$

is expressed in two dimensionless quantities

$$\frac{E_s}{E_m} = \frac{\text{Young's modulus of the slab material}}{\text{Young's modulus of the subgrade}}$$

and

$$\frac{h_s}{a} = \frac{\text{height of the slab}}{\text{radius of the loaded area}}$$

11 "Philosophical Magazine," by A. H. A. Hogg, vol. 25 (1938), p. 576.

12 "Investigations as to the Elastic Properties of Soils and Design of Pavements According to the Theory of Elasticity," by N. Odemark, Statens Väginstitut, Meddelande No. 77, Stockholm, 1949.

Earlier work of E. Reisner[13] and Nijboer[14] has shown that Hogg's relations can be applied to dynamic loading as well. Nijboer demonstrated that the strains calculated from dynamic E-moduli deviate by no more than 16%, on an average, from the strains measured under rolling wheels.

Hogg's relations have several practical consequences:

a. It is not necessary to carry out tests under exactly the same loading conditions (σ_o and a) as occur under traffic circumstances.

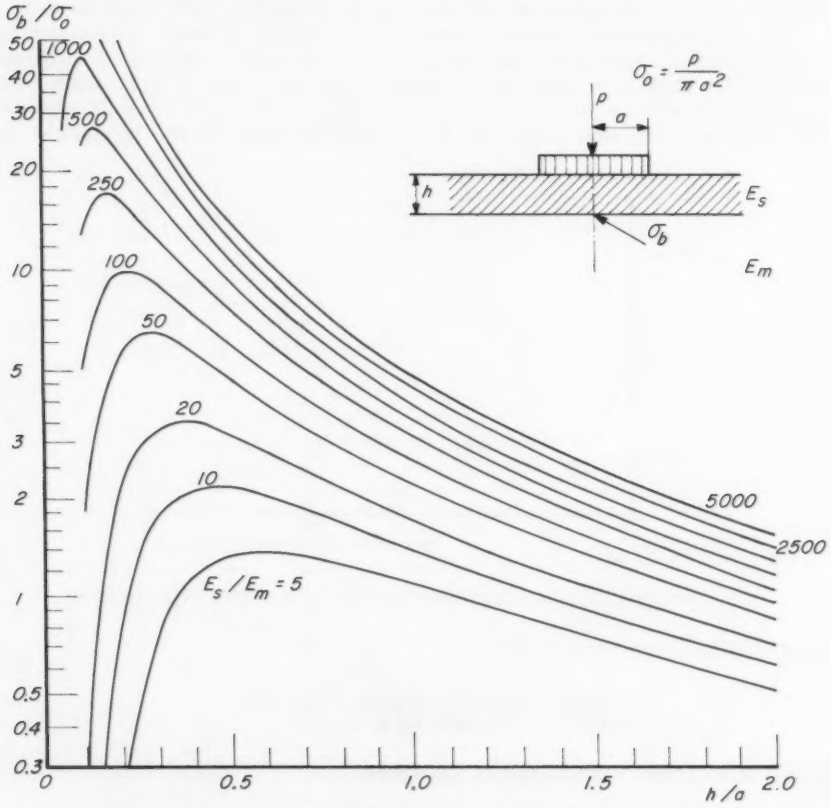


FIG. 5.—BENDING STRESSES AT THE BOTTOM OF A SLAB

b. For the design of a new construction the loading conditions are known beforehand; h_s is to be chosen so that σ_b does not exceed an allowable value which follows from fatigue tests. These calculations can be made if E_m and E_s are known.

c. Data about E_s can be determined in the laboratory, E_m must be measured in the field; this can be done either indirectly by strain measure-

13 "Ing. Archiv," by Reisner, vol. 7 (1936), p. 38.

14 "Ein Berechnungsverfahren für Asphaltstrassen mit besonderer Berücksichtigung des dynamischen Verkehrs (Computation procedure for asphalt roads with particular consideration of dynamic traffic)," by L. W. Nijboer, Die Bautechnik, vol. 33, No. 2 (1956), p. 37.

ments or by any other convenient method for direct determination of of dynamic E-moduli.

In the further investigations described in this paper, use is made of dynamic methods to measure E-moduli in a direct way. The apparatus and the method of measurement are described subsequently.

FIELD MEASUREMENTS WITH SUSTAINED VIBRATIONS

Velocity of Waves Produced by Dynamic Loading.—The stress condition around a load would build up instantaneously if the supporting medium behaves merely elastically. Soils, however, not only show elasticity but also have mass. The existence of mass introduces a delay in the deflections so that the final condition builds up in the course of a short time after a constant static load has been applied. With variable loads like sustained vibrations this leads to the formation of concentric waves propagated away from the loaded area with a velocity given by the ratio of the elasticity to the density d (true density mass/unit of volume) of the soil. If the shear modulus G_m is taken as a measure of the elastic properties, theory shows that the wave velocity V is given by

$$V = p\sqrt{G_m/d} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which p is a factor depending on the nature of the waves. For transversal waves p = 1, for Rayleigh waves it varies between 0.91 and 0.96 depending on Poisson's ratio μ. A survey of the different possibilities has been given, for example, by R. Jones,[15] who, on the basis of the theoretical work of G. F. Miller and H. Pursey,[16] assumes Rayleigh waves to be the most probable.

Young's modulus being related with the shear modulus and the Poisson ratio by

$$E_m = 2\,(1 + \mu)\,G_m \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

it can be derived that E_m is given by

$$E_m = \frac{2\,(1 + \mu)}{p^2}\,d\,V^2 \approx 3\,d\,V^2 \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

From Eq. 4 it appears that the value of the E-modulus can be determined by observing wave velocities. The density plays a minor part as it shows only little variation as compared with the wave velocity which varies between, 50 and 500 m per sec, so that V^2 displays variations by a factor of up to 100 for various materials.

S. G. Bergstrom et al.[17] demonstrated that the dynamic E-modulus derived from velocity measurements on homogeneous soil tallies with the results of plate bearing tests.

---

15 "In-situ measurement of the dynamic properties of soil by vibration methods," by R. Jones, Géotechnique, vol. VIII, No. 1 (1958), p. 1.

16 "On the partition of energy between elastic waves in a semi-infinite solid," by G. F. Miller, and H. Pursey, Proceedings Royal Society (London), series A, vol. 223 (1954), p. 521 and vol. 233 (1955), p. 55.

17 "Dynamic method for determining average elastic properties of surface soil layers," by S. G. Bergstrom, Handlinger Nos. 7 and 13, Svenska Forskningsinstitutet för Cement och Betong vid. Techniska Högskolan i Stockholm, 1946 and 1949.

It will be shown later that there is an excellent correspondence between the dynamic E-modulus of bare soils and the deflections of the loaded area expressed in the so-called dynamic stiffness.

A further discussion of the wave velocity is given subsequently under the heading Dynamic Behavior of Soils.

Deflection at the Top of a Construction.—The deflection under the center of a loaded circular area of an elastic medium has been given by J. Boussinesq.[18] If a uniform static normal stress σ_o is applied to an area with a radius $\underline{a}$ the vertical deflection Z_m amounts to

$$Z_m = \frac{3\,a\,\sigma_o}{2\,E_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

This relation has been extended by Odemark[12] who expressed the deflection Z_s of a single elastic slab on an elastic medium in terms of the deflection which would occur on the elastic medium only, thus

$$Z_s = Z_m\,f = \frac{3\,a\,\sigma_o\,f}{2\,E_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

The factor of f is given by the ratio E_s/E_m and h_s/a and has been presented in a graphical form, in Fig. 6.

The force exerted upon the whole loaded area is

$$F = \pi\,a^2\,\sigma_o \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

The elastic stiffness R of the construction is defined as the ratio between force and deflection. This quantity can be compared with a spring constant. It follows from Eqs. 6 and 7 that on the bare medium

$$R_m = \frac{F}{Z_m} = \frac{2\pi}{3}\,a\,E_m \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

and on the elastic slab

$$R_s = \frac{F}{Z_s} = \frac{2\pi}{3}\,a\,E_m/f \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

These equations imply the following consequences:

a. R_m is a direct measure of the E-modulus of a subgrade. If R_m is observed on a stratified subgrade, an average value is found for the moduli of the strata, which represents the effective average under road conditions.
b. The ratio of the elastic stiffnesses R_s/R_m measured on the pavement and on the subgrade, respectively, gives Odemark's factor f which is a measure of E_s/E_m and thus of the E-modulus of the pavement. The same procedure can be followed if the value of E_m is known from velocity measurements and R_s is observed on the pavement. In this case the Eq. 9 is used to find the value of f.
c. Actual soils not being ideally elastic, the factor $2\pi/3$ may have to be replaced by a somewhat different factor.

18 "Application des potentiels, à l'étude de l'équilibre et due mouvement des solides élastiques," by Gauthier-Villars, Paris, 1885.

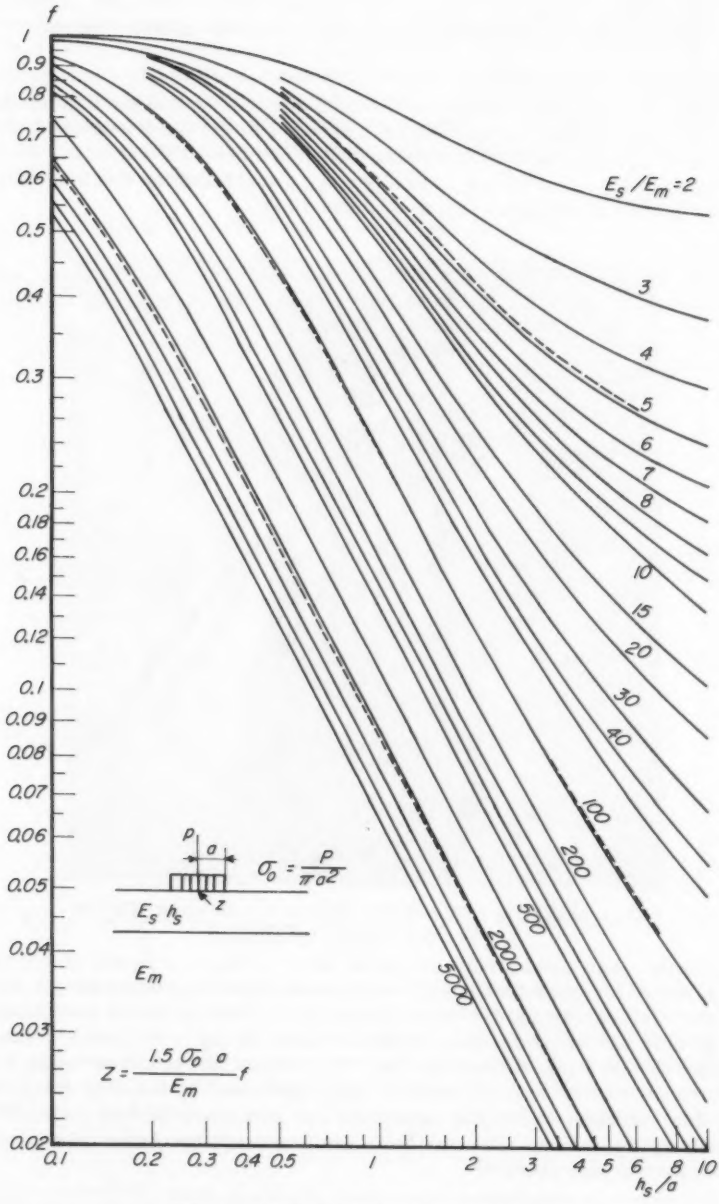


FIG. 6.—DEFLECTION OF A PAVEMENT

d. Actual soils possessing mass, the dynamic stiffness is different from the elastic stiffness. It may be mentioned that the value of the elastic stiffness can be derived from dynamic stiffness measurements.

Experimental Technique.—

Vibrators. Different vibrators are used for the low and the high frequency range. At low frequencies up to 600 cycles per sec the vibrations are produced by a heavy mechanical vibrator described by Nijboer and van der Poel.[19] At high frequencies up to 10,000 cycles per sec a much lighter electromagnetical system is used which gives forces of about 10 kg.

FIG. 7.—VIBRATOR FOR WAVE VELOCITY MEASUREMENTS AT HIGH FREQUENCIES

A vibrator was used giving forces of about 10 kg at a power of 100 watts. This suffices for high frequencies because the corresponding waves remain near the surface and only a small mass of soil has to be set into vibration. The vibrator has an additional weight of about 50 kg and "floats" over the pavement on three soft rubber blocks. The moving coil is connected to a light bar which is pressed onto the surface of the pavement by a spring leaf. Fig. 7 shows this vibrator. A similar apparatus has been described by Jones.[20]

Auxiliary Equipment. The auxiliary equipment for the heavy vibratory has been mentioned elsewhere.[19]

---

19 "A Study of Vibration Phenomena in Asphalt Road Construction," by L. W. Nijober, Proceedings, American Association of Asphalt Paving Technologists, vol. 22 (1953), pp. 197-331.

20 "A vibration method for measuring the thickness of concrete road slabs in situ," by R. Jones, Magazine of Concrete Research, vol. 7, No. 20 (1955), p. 97.

The high frequency vibrator is excited by a 100-watt power amplifier which gets its input signal from the standard R. C. oscillator. Fig. 8 shows the arrangement. The phase mark is obtained by cathode ray suppression so that there is a short interruption of the sine on the screen of the oscilloscope.

Measurement of the Wave Velocity. Waves require a time x/V to travel over a distance x through soil in which the wave velocity is V. Thus, if the phase condition of the vibrations under the center of the vibrator is described as $\sin \omega t$, the corresponding condition at a distance x from the center is given by

$$\sin \omega (t - x/V) = \sin (\omega t - 2\pi \frac{\nu x}{V}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which ν is the frequency in cycles per sec and $\omega = 2 \pi \nu$ is the angular frequency. According as a pick-up is moved away from the vibrator, the signal

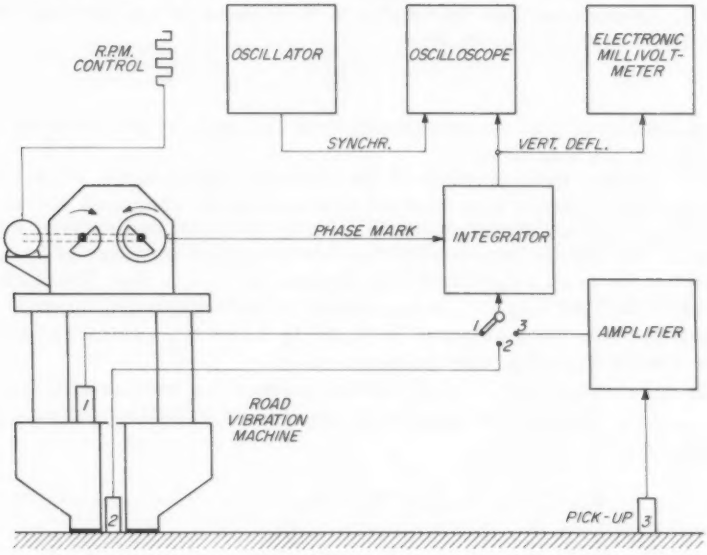


FIG. 8.—BLOCK DIAGRAM SHOWING THE ROAD VIBRATION MACHINE AND AUXILIARY APPARATUS

will show a steadily increasing phase shift. The phase mark, however, appears on the screen of the oscilloscope at constant phase condition. Consequently, the sine and the phase mark are gradually shifted with respect to each other as the pick-up is moved away. When the phase mark is at the trough of the sine for x = 0, it will move towards the crest of the sine first, and at a certain distance x_1 it will be at the next trough. This corresponds with a phase of 360 degrees or 2π radians. Thus

$$\frac{\nu x_1}{V} = 1 \text{ or } x_1 = \frac{V}{\nu} = L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which L is the wave length. In the same way places can be found at longer distances x_2, x_3, and others, where the phase mark is at the trough of the sine

again, corresponding with 2L, 3L, and others. In order to find the wave length L, it suffices theoretically to measure the distance between two adjacent places where the same phase conditions occur. More accurate results are obtained when several observed distances such as x_1, x_2 and x_3 are plotted as a function of the number of waves. The result will usually be a straight line, from the slope of which the wave length can be determined with far greater accuracy. The wave velocity then follows from $V = \nu L$.

Half wave lengths can be observed if in addition the distances $x_{1/2}$, $x_{1-1/2}$, etc., are noted where the phase mark is at the crest of the sine. With the heavy vibrator it is possible to switch off the integrator so that quarter waves can also be observed.

Measurement of the Dynamic Stiffness. The dynamic stiffness of a pavement has been defined by van der Poel[21] as the ratio between the amplitude of the vertical force F which is applied to the pavement over a circular area of 30 cm is diameter and the amplitude $\hat{Z}$ of the resulting vertical deflection under the center of the load, thus

$$S = \hat{F}/\hat{Z} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

It is measured in correspondence with this definition by simply observing the deflection at a known force.

For a proper understanding of the dynamic stiffness, the combination of road vibration machine plus road construction can be compared with a system consisting of purely elastic springs, damping dashpots, and masses, as shown in Fig. 9. The elastic behaviour of the construction is represented by a series of springs $R_1 \ldots\ldots R_n$ bearing the masses $M_1 \ldots\ldots M_n$. The springs are damped by dashpots $A_1 \ldots\ldots A_n$. Owing to its sturdy build, the road vibration machine can be considered as a single mass M_m, which has a damped elastic contact $R_m + A_m$ with the pavement.

The movements $Z_1 \ldots\ldots Z_m$ of the masses are related with the forces $F_1 \ldots\ldots F_m$ between the masses by two series of differential equations of the type

$$F_k = \left(R_k + A_k \frac{d}{dt}\right)(Z_k - Z_{k-1}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

and

$$F_{k+1} = F_k + M_k \frac{d^2 Z_k}{dt^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

This system is to be considered for harmonic forces with an angular frequency $\omega = 2\pi\nu$.

The final solution for sustained vibrations shows that the deflection of the pavement lags in phase with respect to the harmonic force. Physically speaking, the relation between force and deflections, as given by the dynamic stiffness S, is not complete unless the phase lag ϕ is also given.

This can be demonstrated by considering a simplified model of a vibrating pavement consisting of one spring R, one mass M, and one cashpot A only. The dynamic stiffness is

$$S = \sqrt{(R - \omega^2 M)^2 + \omega^2 A^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

21 "Journal Applied Chemistry," by C. van der Poel, vol. 1 (1951), p. 281.

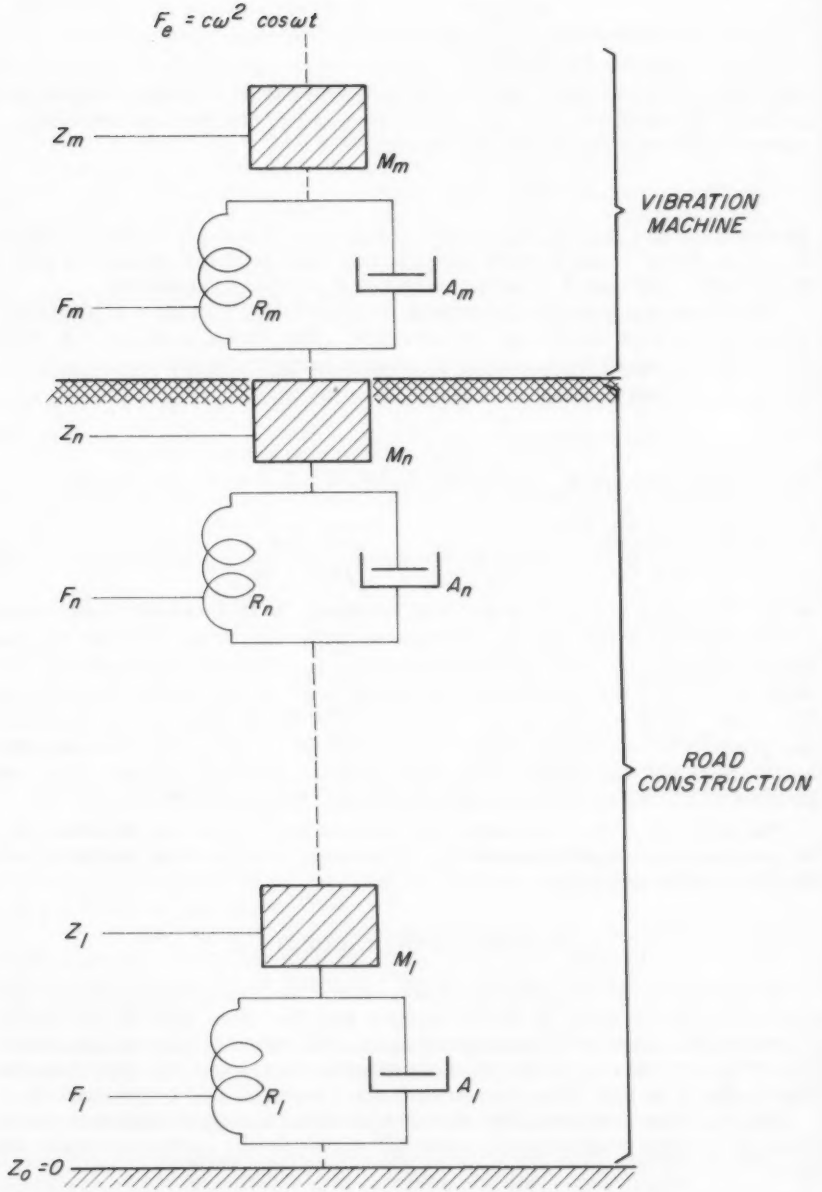


FIG. 9.—REPRESENTATION OF THE MECHANICAL BEHAVIOUR OF VIBRATING SOIL BY SPRINGS, DASHPOTS AND MASSES

and the phase angle ϕ between force and deflection is given by

$$\tan \phi = \frac{\omega A}{R - \omega^2 M} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

The Eqs. 15 and 16 show that both S and ϕ depend on frequency. The elastic stiffness R, which is the most important quantity, can be found from experimental S-values by using the relationship

$$S \cos \phi = R - \omega^2 M. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

As appears from Eq. 17, the elastic stiffness R is found by linear extrapolation in a plot of S cos ϕ versus ω^2. To this end the phase angle ϕ as well as the dynamic stiffness S must be measured at several frequencies.

The phase angle is also important for calculating the force F_m which the vibration machine exerts on the pavement. This force is different from the force F_e generated by n eccentric masses having a weight w and an eccentricity r, so that the amplitude of the force fs

$$F_e = n\, w\, r\, \omega^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

The relation between F_m and F_e is given by van der Poel's equation[22]

$$\left[\frac{\hat{F}_m}{\hat{F}_e}\right]^2 = 1 + 2\frac{Z_m}{c}\cos \phi_m + \left[\frac{Z_m}{c}\right]^2 \quad \ldots\ldots\ldots \quad (19)$$

in which $c = n\, w\, r/M_m$ is a machine constant. In this case it is necessary to measure the phase lag ϕ_m between the deflection of the machine Z_m and the eccentric force. The measurement of the force on the pavement thus consists in reading the voltage and the phase angle of the signal from pick-up No. 1, and the frequency of the vibrator. The phase angle ϕ_m is read from the graduated arc after the phase marker has been adjusted so that the phase mark appears at the trough of the sine on the oscilloscope screen. The computation of F_m is facilitated by nomographical representation[23] of Eq. 19.

The same procedure repeated with pick-up No. 2 gives the deflection Z of the pavement and data to compute the phase angle ϕ between the deflection and the force on the pavement.

DYNAMIC BEHAVIOR OF SOILS

Wave Velocity in Homogeneous Soils.—A soil is homogeneous, from a mechanical point of view, if its E-modulus has the same value at all depths. Consequently waves will be propagated with the same velocity at all depths. Low-frequency waves, which penetrate deeply into the soil, will thus have the same velocity as high-frequency waves, which remain close to the surface.

This has been confirmed by various experiments, among which was one on a fill 18 m deep, containing a compacted homogeneous mixture of loam and marl.

---

22 "Vibration Research on Road Constructions," by C. van der Poel, ASTM Special Technical Publication No. 156, (1954), p. 174.

23 "Philips Sci Ind," L. W. Nijboer, vol. 2, (1955), p. 1.

Table 1 gives a brief survey of wave velocities observed on various types of soil, and the E_m values computed from these velocities.

The E_m values depend primarily on the soil strength. Strength varies with soil type, degree of compaction, water content, and previous treatment. Clay can show very low values in the saturated condition.

Dynamic Stiffness of Homogeneous Soils.—The dynamic mechanical behaviour of homogeneous soils has been represented by a simple model consisting of one spring R, one mass M, and one damping element A. Such a representation is correct if all the quantities R, M, and A have values which are independent of frequency. This is found to be true at frequencies low enough for the wave length to be large as compared with the radius of the body of soil in which the greater part of the deformation takes place. All the distributed mass particles then show unidirectional movements and give a positive contribution to the total mass effect. The model may be used with a constant mass M_o under these circumstances.

At relatively high frequencies, however, the wave length becomes rather short and parts of the distributed mass of soil vibrate in opposite directions.

TABLE 1.—WAVE VELOCITIES AND E_m VALUES OBSERVED ON VARIOUS SOILS

Type of Soil	V, in meters per second	E_m, in Kilograms per square second
(1)	(2)	(3)
Peat	50-110	80-330
Clay	90-150	440-1200
Sand	120-180	800-1800
Sandy clay	150-200	1200-2200
Clay gravel	190-320	2100-6000
Moraine	250-400	3500-9000

It has been found that this condition can still be represented by a three-element model provided a value inversely proportional to the third power of frequency is taken for the mass.

Fig. 10 shows the experimental points found for the dynamic stiffness and the phase angle on a homogeneous fill 18 m thick. The solid line shows the theoretical curve for constant mass, the dotted line for variable mass. Two conclusions can be drawn:

a. The constant mass at low frequencies enables us to determine the elastic stiffness by linear extrapolation in a plot of S cos ϕ versus ω^2 (or ν^2) as shown in Fig. 11.
b. The phase angle is small as compared with 90 degrees.

It has been found that these conclusions are valid for all soils and pavements, except for cases involving fills placed on extremely soft clay and peat.

The extrapolated R values tally very well with the dynamic E_m value found from velocity measurements in a frequency range between about 40 and 400 cycles per second. Fig. 12 shows the results obtained on 10 different soils.

The average straight line corresponds with

$$R = 2.5\, a\, E_m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

The factor of 2.5 replaces the theoretical factor of $2\pi/3 = 2.1$ which would hold for ideal elastic substances. The vibration amplitude has been found to decrease more rapidly with increasing depth in actual soils than it would do in ideal Boussinesq substances. This discrepancy is of the right order of magnitude to account for the difference of about 20% in the stiffness factor. Moreover, it indicates that under this type of loading the main part of the de-

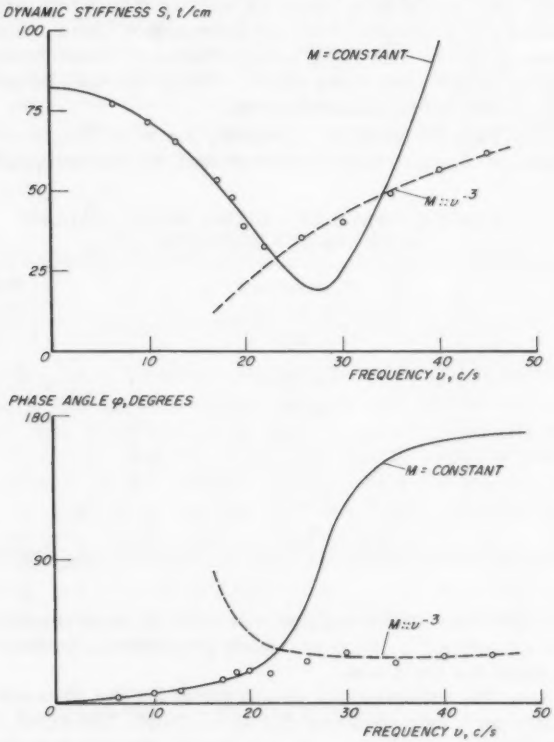


FIG. 10.—DYNAMIC STIFFNESS S AND PHASE ANGLE ρ AS A FUNCTION OF FREQUENCY

formation in the soil is restricted to a depth equal to some five to ten times the radius of the loaded area.

Velocity Measurements on Stratified Soils.—The wave velocity observed on stratified soil, for example, a base course on subgrade, depends on the frequency at which the measurement is carried out. At frequencies so high that the wave length L is small as compared with the thickness h of the top layer, the waves are propagated through this layer only. A uniform wave velocity is found, corresponding with the E-modulus of the top layer material. At very low frequencies, however, the waves penetrate into the soil so deeply that the

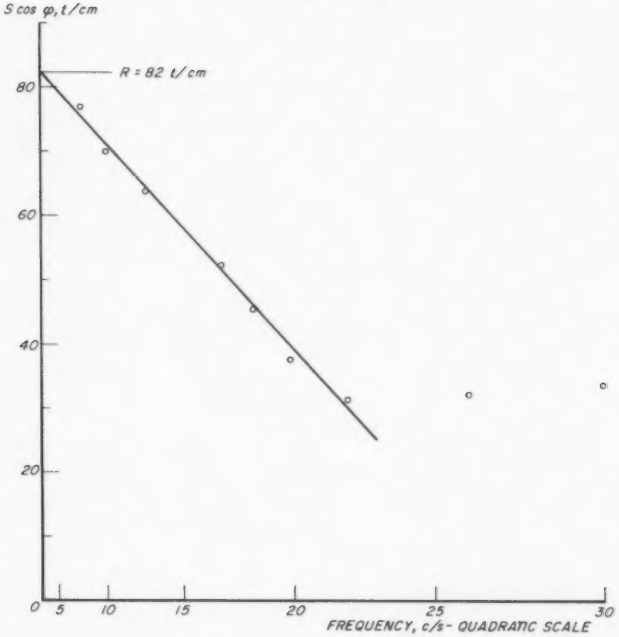


FIG. 11.—LINEAR RELATIONSHIP BETWEEN S cos ρ AND THE SQUARE OF THE FREQUENCY USED FOR THE DETERMINATION OF THE ELASTIC STIFFNESS R

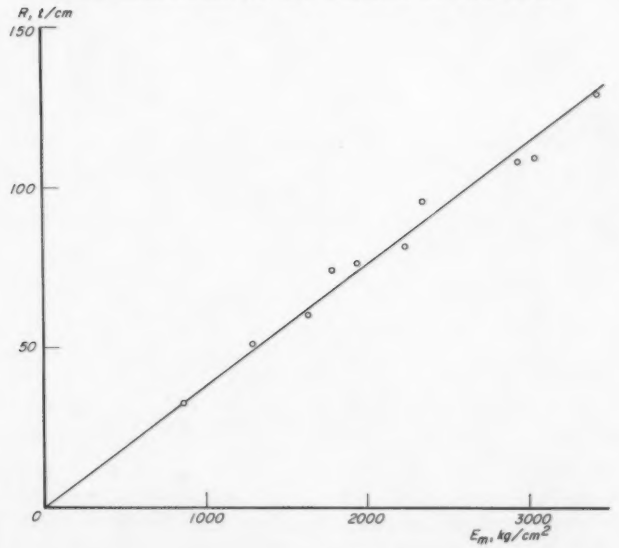


FIG. 12.—RELATION BETWEEN THE ELASTIC STIFFNESS R AND THE DYNAMIC E_m-MODULUS OF HOMOGENEOUS SOILS

wave velocity depends on the E-modulus of the deep soil only. The transition from one wave velocity to the other takes place in a frequency range on which the theory of Rayleigh waves gives some information.[15] The results of our measurements indicate that the transition occurs at practically constant wave length which equals about twice the thickness of the top layer. In other words, if the wave velocity V is plotted as a function of half the wave length, an impression is obtained of the rigidity of the soil as a function of depth.

Fig. 13 gives an example for three layers. Fig. 13(a) shows the relation between the wave velocity and the frequency. Three practically constant

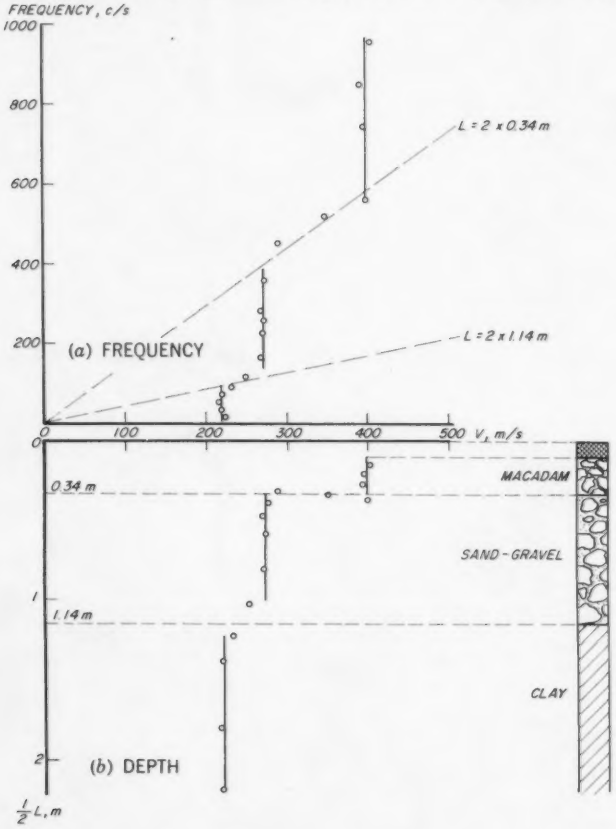


FIG. 13.—WAVE VELOCITIES OBSERVED ON STRATIFIED SOIL, AS A FUNCTION OF FREQUENCY AND DEPTH

values are found in different frequency ranges. The transition is indicated by lines for wave lengths equal to two times the exact depths of the interfaces. Fig. 13(b) shows the wave velocity as a function of half the wave length (approximate depth) in comparison with a cross section of the construction (exact depth). The identification of the layers to which the observed wave velocities must be attributed is facilitated by this type of graph. It does not, however, provide means for determining the exact depth of interfaces. Methods for this purpose have been described by Jones.[15]

Another example is given in Fig. 14. Unlike the preceding instance the top layer (clay) here is softer than the deep soil (moraine). Under these circumstances there is a frequency range - here between 30 and 50 cycles per second - where two wave velocities can occur at one frequency. The two wave velocities have indeed been found. Fig. 15 shows the propagation curve at 35 and 45 cycles per second. The lower velocity belonging to the top of the soil is found close to the vibrator, and the high velocity belonging to the deep soil at longer distances from the vibrator. Similar results have been obtained for various types of soils. In some cases two wave velocity levels have been found, one for the deep soil and the other for the top. This points to the coexistence of separate waves in separate strata, a result which cannot be explained theoretically[15] unless transversal waves are assumed to occur.

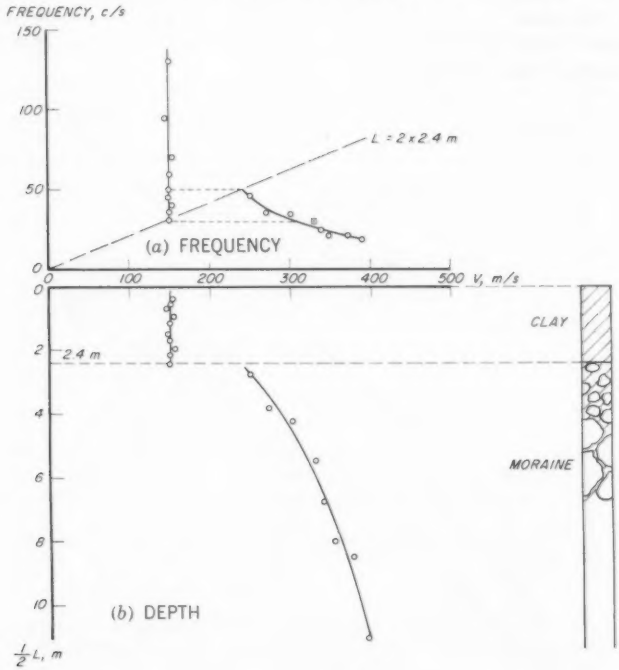


FIG. 14.—WAVE VELOCITIES OBSERVED ON STRATIFIED SOIL, AS A FUNCTION OF FREQUENCY AND DEPTH

E-moduli of Base Courses.—The E-moduli of base courses have been derived from wave velocity measurements and also from the elastic stiffnesses observed on both base course and subgrade. The results obtained with these two methods show a good agreement.[24] Nijboer[14] derived E-moduli of base courses from dynamic stiffness measurements on pavements and low-frequency wave velocity measurements in the subgrade.[25]

---

24 Wegen, L. W. Nijboer and W. Heukelom, vol. 32 (1958), p. 108.
25 "Unpublished Measurements in Germany," 1957.

It has been found that the compaction and thus the value of E_1 of a base course depends largely on the value of E_m of the subgrade. The ratio E_1/E_m is mostly between 1.5 and 2.5 for the materials marked with an asterisk in the following tabulation of dynamic E-moduli of base course material. The E-modulus of tar- and bitumen-bound materials depends on temperature and binder content in addition to the characteristics of the aggregate blend.

Type	E_1, in kg per sq cm
Hand-pitched	3,000-5,000
Macadam*	1,600-6,700
Stabilized sand	4,950-5,200
Gravel-sand*	4,500-8,200
Tar-bound macadam	17,500-32,000
Bitumen-bound sandy gravel	25,000-100,000

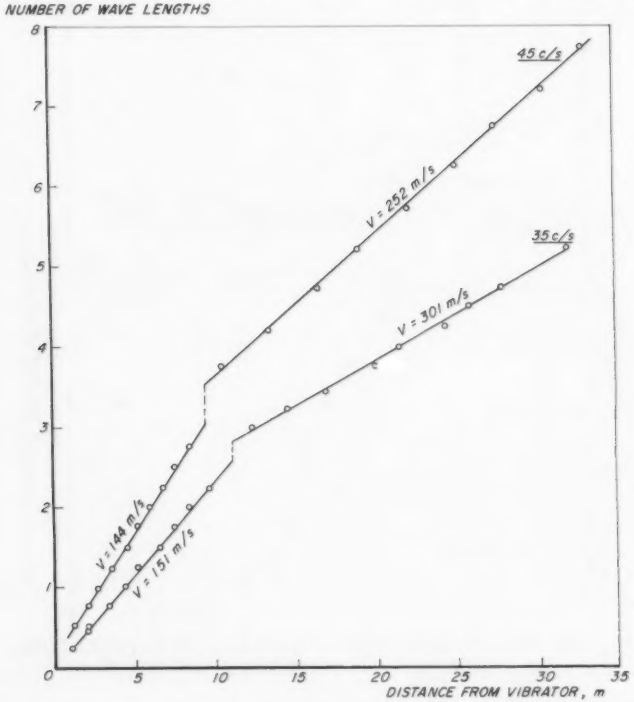


FIG. 15.—WAVE PROPAGATION CURVES OBTAINED ON THE SOIL OF FIG. 14.

Relation Between the Dynamic E-modulus and the CBR-value.—The CBR-value is frequently used to express the strength of soils and base course materials and the reader will be interested in the relation between the CBR-value and the dynamic E-modulus.

By introducing the diameter of the CBR piston in the Boussinesq equation for the surface deflection under the center of a uniform circular load, and by assuming that Poisson's ratio is 0.5 and the CBR at 0.1-in. deflection equals

the applied load on the piston in lbs. per sq. in. divided by 1000, it can be shown that the CBR-value of an elastic Boussinesq substance is related to the E-modulus (in kg per sq cm) according to E = 10 CBR. Actual soils, however, behave differently from elastic substances so that, for instance, after deformation under the CBR piston the soil does not completely rebound on removal of the load. A considerable part of the deformation is plastic and only a small percentage of the deflection is elastic.

Under dynamic loading conditions the elastic deformation is recorded exclusively. The dynamic E-modulus is accordingly found to be higher than 10 CBR. The ratio between plastic and elastic deformation varies for various soils so that correlation between E and CBR shows a considerable scatter.

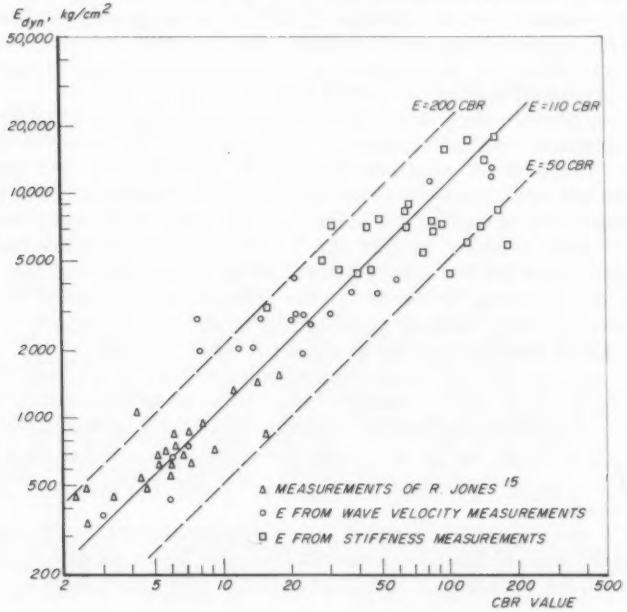


FIG. 16.—RELATION BETWEEN THE DYNAMIC MODULUS ELASTICITY AND THE CBR VALUE OF VARIOUS SOILS

Fig. 16 shows this correlation. It has been found that, on an average

$$E_{dyn} = 110\ CBR \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

The factor varies between about 50 and 200 for individual soils.

DYNAMIC BEHAVIOUR OF PAVEMENTS

Traffic Stiffness.—The dynamic stiffness T under traffic conditions is not equal to the dynamic stiffness S measured with sustained vibrations. Traffic loads a pavement only once per wheel and this load in its dependence on time more or less resembles the first half sine of a sustained vibration. This initial phenomenon cannot be recorded with the road vibration machine from which readings are obtained after a number of deflections.

A mathematical approach of the initial phenomenon is possible because it has been found that on good subgrades the phase angle ϕ is small as compared with 90°. This results in a considerable simplification of the problem, as for $\phi = 0$ the traffic stiffness T must be equal to the elastic stiffness R. It has been derived that this equality remains almost exact for small phase angles, so that we may put

$$T \approx R \pm 20\% \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

The moving mass M and the damping A in this case have hardly any influence on the traffic stiffness and thus on the stresses which are generated in a pavement on good subgrade under traffic conditions. Consequently the stresses and the strains can be calculated only from the elastic stiffness R, and the use of conventional theories on elastic constructions, which do not account for mass and damping, is allowed. It has not yet been ascertained whether this holds for constructions on peat as well.

Equivalent Slabs.—Hogg and Odemark[11, 12] treat the problem of a single slab on a subgrade. Roads and similar constructions are, however, built up of two or more layers. A complete theoretical treatment of this complicated case has not yet been given. Nijboer[10,14] has fitted the problem to the theory by the introduction of equivalent slabs. In this treatment use is made of two alternative types of slabs, sI and sII. The equivalent slabs are computed on the assumption that the two layers are subjected to uniform bending.

Slab sI has the same thickness as the actual pavement and a modulus E_{sI} that makes the bending moment equal to that of the real pavement. It has been used as an aid in determining the E-moduli of base courses.

Slab sII has not only the same bending moment, but also a neutral fiber at the same depth as the actual construction. This can be obtained by a suitable choice of the thickness h_{sII} and the modulus E_{sII}. Such an equivalent slab has the same strain at the top as the actual construction and it has been successfully used for computing strains, which could be compared with directly measured ones.

Nijboer's method can be extended to any number of layers. To this end the depth D of the neutral fiber is computed from the Young moduli E_i, the thicknesses h_i, and the depths z_i of the centers of the respective layers:

$$D = \frac{\Sigma E_i h_i z_i}{\Sigma E_i h_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

According to its definition the thickness of slab sII is

$$h_{sII} = 2 D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

The distance of the interfaces from the neutral fibre is denoted by r_i. It is found that

$$E_{sII} = 1/2 \left[E_i + (E_2 - E_1) \left(\frac{r_1}{D}\right)^3 + \ldots\ldots - E_n \left(\frac{r_n}{D}\right)^3 \right] \ldots\ldots \quad (25)$$

Strain at the Top of Pavements.—The strain at the top of pavements can be derived from the radius of curvature of the pavement under a load. Odemark[12] has derived for the radius of curvature

$$R_o = \frac{4 E_s a}{3 \sigma_o} F_R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (26)$$

where F_R is a dimensionless quantity which depends on E_{sII}/E_m and h_s/a. Odemark further indicates that for a homogeneous soil the radius of curvature R_z at a depth z follows from

$$R_z = R_o \frac{[1 + (z/a)^2]^{5/2}}{1 + 4\,(z/a)^2} = \frac{R_o}{\alpha} \quad \ldots\ldots\ldots\ldots\ldots \quad (27)$$

Assuming that Eq. 27 also holds for a layered system, Nijboer[8] calculated the strain at the top for two-dimensional deformation and obtained, again in dimensionless quantities,

$$\frac{\epsilon\, E_m}{\sigma_o} = \frac{3}{8} \frac{h_{sII}}{a} \frac{\alpha}{F_R} \frac{E_m}{E_{sII}} \quad \ldots\ldots\ldots\ldots\ldots \quad (28)$$

The term on the left-hand side of Eq. 28 can be expressed in h_{sII}/a and E_s/E_m and is represented in this way in Fig. 17.

The strains computed with Eq. 28, using experimental values of the dynamic E-moduli, tally with those measured with strain gauges as shown in Fig. 18. There is some scattering of the individual points, but on an average the computed strains deviate by no more than 7% from the observed values. Most of the calculations have been made with the aid of equivalent slab sII.

Under the conditions of heavy road traffic, a suitable shock coefficient must be taken into account. There is also a difference between the static and the dynamic contact of tires with a pavement, which must be corrected for. The dynamic contact pressure is about 33% higher and the radius of the contact area about 16% smaller than under static conditions.

The E-modulus of bituminous pavements depends on the temperature and, as a result, so does the strain. Fig. 19 shows the computed stress (for a 2 in. a.c. on a 6 in. and 10 in. base course) at the top of a construction under a given load as a function of the temperature. Fig. 19 demonstrates that the stresses at low temperatures are much higher than those at high temperatures.

The probability of fatigue failures can be derived from these stresses and data concerning the fatigue properties of the paving material. For an accurate prediction of the most probable lifetime of a pavement it is also necessary to consider the local yearly temperature distribution curve, the traffic intensity, the degree of channelization and other conditions. The reliability of such a prediction depends largely on the available fatigue data of bituminous paving materials. This part of the investigation has not yet been completed. The results obtained to date[26] are promising.

DYNAMIC LABORATORY MEASUREMENTS

A survey of experimental techniques would not be complete without a brief description of the laboratory measurements concerning the dynamic E-modulus and the fatigue strength of paving materials.

Both the E-modulus and the fatigue strength are measured in a wide range of temperatures and loading times (frequencies). The measurements are carried out in a four-point bending machine driven by an eccentric, the stroke of

26 "Fatigue of bituminous road mixtures," by R. N. J. Saal and P. S. Pell, Paper to be presented at the British Society of Rheology Meeting, January, 1959.

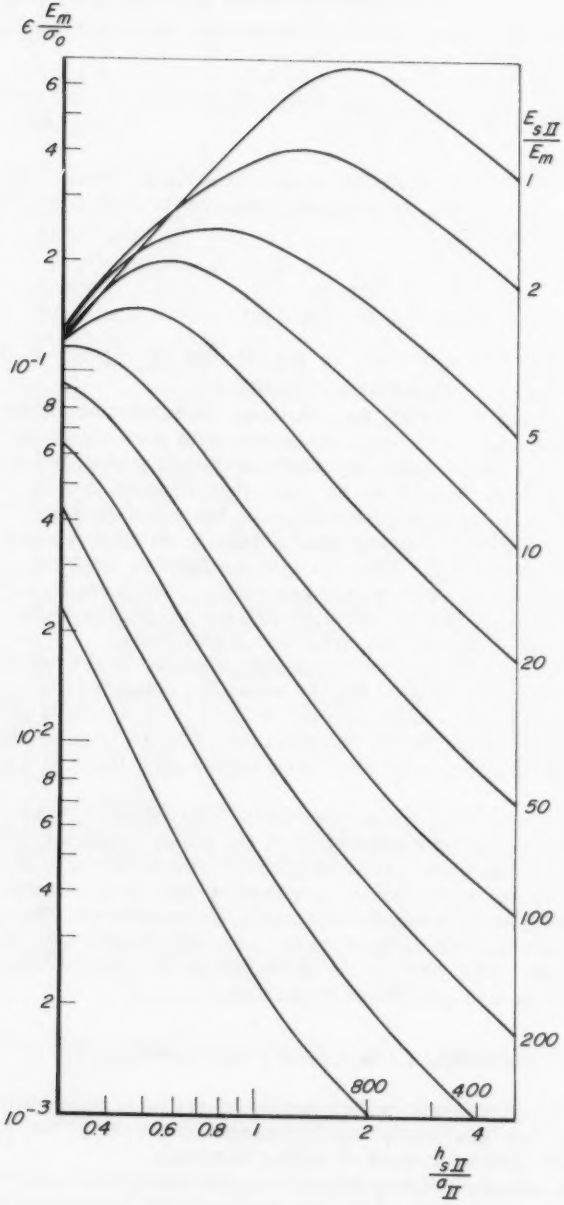


FIG. 17.—STRAIN AT THE TOP OF A PAVEMENT ACCORDING TO NIJBOER

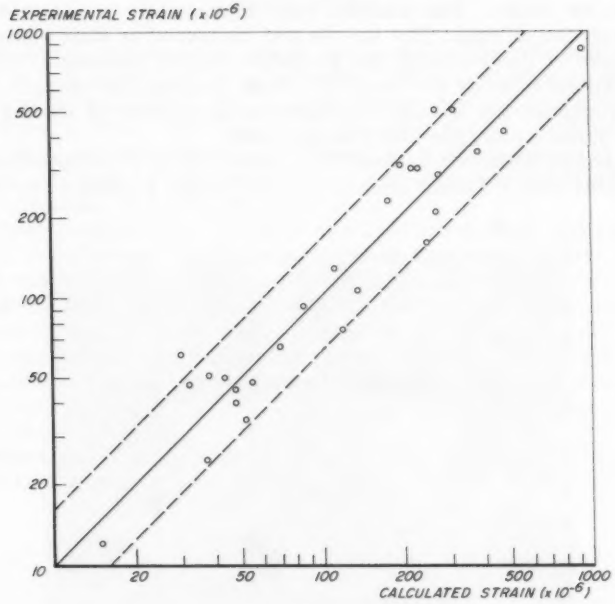


FIG. 18.—CORRELATION BETWEEN EXPERIMENTAL AND CALCULATED STRAINS

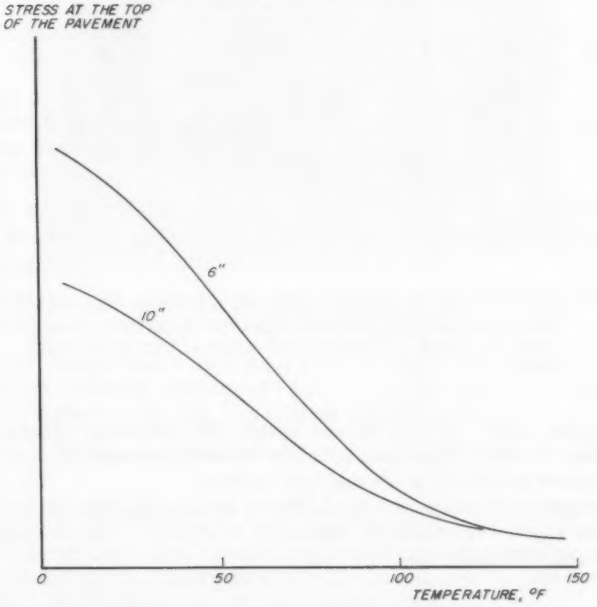


FIG. 19.—STRESS AT THE TOP OF A PAVEMENT

which can be varied. The machine runs continuously and exerts sinusoidal forces upon the sample. The forces are measured in such a way that, for a given stroke of the machine, the E-moduli can be computed. The samples have thicknesses which are usual for road carpets, for example, 2 in. The fatigue properties are studied by observing the number of loading cycles required to break the sample under known loads.

The dynamic E-moduli so found show an excellent correspondence with the experimental data obtained from tests with beams vibrating freely. The fatigue strengths tally with the values found with revolving cylindrical specimens bearing a radial load and with the results obtained with a three-point bending machine driven by a moving coil system.

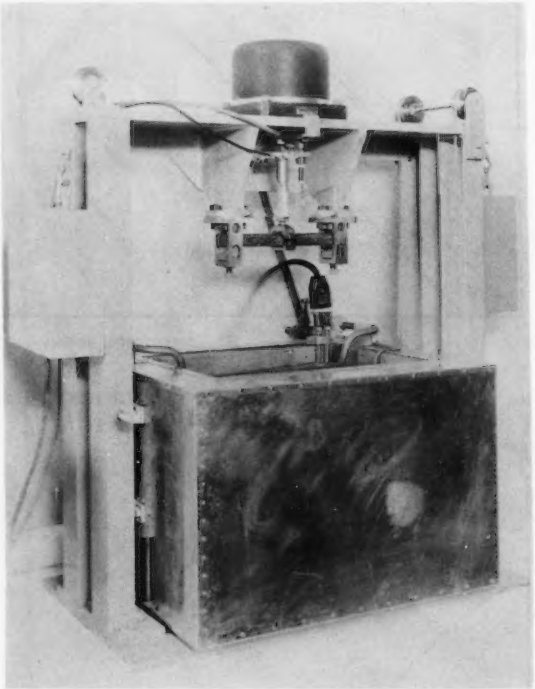


FIG. 20.—THREE-POINT BENDING MACHINE FOR RAPID DETERMINATION OF THE DYNAMIC E-MODULUS AND THE FATIGUE STRENGTH OF ASPHALTIC MIXTURES

The three-point bending machine, shown in Fig. 20, has proved its practical value for rapid evaluation of asphaltic mixtures. The samples can be as small as 1-1/2 × 2 × 20 cm.

CONCLUSIONS

Dynamic testing of pavements offers many interesting possibilities. One of the main advantages of this nondestructive testing method is its close correspondence with the dynamic character of moving traffic. The use of a single frequency vibrator (which produces monochromatic waves), provides a means of obtaining detailed information about the mechanical properties of underlying strata, which cannot be gained with nondestructive static loading tests or strain measurements on the top of a pavement.

The present paper gives a survey of the lines along which dynamic testing has been developed in the Netherlands. Stiffness and wave velocity measurements were used to collect data concerning the dynamic mechanical properties of base courses and subgrades. These data served to compute strains and stresses in the pavement under moving vehicles, which could be compared with the fatigue strength of the paving materials. It is demonstrated that the computed strains tally with the strains observed under a moving test vehicle. From this it is concluded that the underlying theoretical considerations are correct.

The present paper gives a few data concerning the dynamic properties of various construction materials and the relation with the corresponding CBR values.

REFERENCES

3. "Strasse und Autobahn," by L. W. Nijboer, vol 8. (1957), p. 210.

4. "Manual on Hot-mix Asphaltic Concrete Paving," by W. Smith, Construcstruction Series No. 72, The Asphalt Institute, 1950.

5. "Plasticity as a Factor in the Design of Dense Bituminous Road Carpets," by L. W. Nijboer, Elsevier Publishing Co., Inc., Amsterdam, 1948.

6. "Rheology and Applied Mechanics," by R. N. J. Saal and C. Van der Poel, "Representation of rheological properties of bitumens over wide range of temperatures and loading times." Proceedings, Second International Congress on Rheology, Oxford (July, 1953), pp. 28 and 331.

7. "General System Describing Visco-Elastic Properties of Bitumens and and its Relation to Route Test Data," by D. van der Poel, Journal of Applied Chemistry, vol. 4, part 5 (May 1954), p. 221.

8. Chapter IX of: "M. Reiner, Builder Materials, Their Elasticity and Inelasticity," by C. van der Poel, Amsterdan, 1955.

9. "Time and Temperature Effects on Deformation of Asphaltic Bitumens and Bitumen-Mineral Mixtures," by C. van der Poel, Journal, Society Plastics Engineers, vol. 11, No. 7 (Sept 1955), p. 47.

10. "Rev Gén. Routes et Aérodromes," by L. W. Nijboer, vol. 27, No. 302 (1957), p. 63.

11. "Philosophical Magazine," by A. H. A. Hogg, vol. 25 (1938), p. 576.

12. "Investigations as to the Elastic Properties of Soils and Design of Pavements According to the Theory of Elasticity, by N. Odemark, Statens Väginstitut, Meddelande No. 77, Stockholm, 1949.

13. "Ing. Archiv," by E. Reisner, vol. 7 (1936), p. 38.

14. "Ein Berechnungsverfahren für Asphaltstrassen mit besonderer Berücksichtigung des dynamischen Verkehrs (Computation procedure for asphalt roads with particular consideration of dynamic traffic)," by L. W. Nijboer, Die Bautechnik, vol. 33, No. 2 (1956), p. 37.

15. "In-situ Measurement of the Dynamic Properties of Soil by Vibration Methods." by R. Jones, Géotechnique, vol. VIII, No. 1 (1958), p. 1.

16. "On the Partition of Energy Between Elastic Waves in a Semi-infinite Solid," by F. G. Miller and H. Pursey, Proceedings Royal Society (London), series A, Vol. 223 (1954), p. 521 and vol. 233 (1955), p. 55.

17. "Dynamic Method for Determining Average Elastic Properties of Surface Soil Layers," by S. G. Bergstrom, et col., Handlinger Nos. 7 and 13, Svenska Forskningsinstitutet för Cement och Betong vid. Techniska Högskolan i Stockholm, 1946 and 1949.

18. Application des potentials, à l'étude de l'équilibre et due mouvement des solides élastiques, by J. Boussinesq, Gauthier-Villars, Paris, 1885.

19. "A Study of Vibration Phenomena in Asphalt Road Construction," by L. W. Nijboer and C. van der Poel, Proceedings, American Association of Asphalt Paving Technologists, vol. 22 (1953), pp. 197-231.

20. A Vibration Method for Measuring the Thickness of Concrete Road Slabs in Situ," by R. Jones, Magazine of Concrete Research, vol. 7, No. 20 (1955), p. 97.

21. "Journal Applied Chemistry," by C. van der Poel, vol. 1 (1951), p. 281.

22. "Vibration Research on Road Constructions," by C. van der Poel, ASTM Special Technical Publication No. 156, (1954), p. 174.

23. Philips Sci Ind, by L. W. Nijboer, vol. 2, (1955), p. 1.

24. Wegen, by L. W. Nijboer and W. Heukelom, vol. 32 (1958), p. 108.

25. "Unpublished Measurements in Germany," 1957.

26. "Fatigue of Bituminous Road Mixtures," by R. N. J. Saal and P. S. Pell, Paper to be presented at the British Society of Rheology meeting, January, 1959.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

ANCHORED BULKHEAD DESIGN BY NUMERICAL METHOD

By F. E. Richart, Jr.,[1] M. ASCE

SYNOPSIS

Newmark's numerical method for solving beam problems was used in the classical design procedures for anchored bulkheads. Direct solutions were obtained rapidly and easily by this method, as illustrated by examples.

INTRODUCTION

The classical design methods for anchored bulkheads involve a consideration of the stability of the bulkhead under the imposed soil loads without consideration of the bending flexibility of the piles. Since their development by R. A. Pennoyer,[2] these methods have been used continuously in design offices for the selection of sheet pile lengths and section properties. The three methods commonly used are (a) the "Free Earth Support," (b) the "Elastic Line," and (c) the "Equivalent Beam" methods: each method has been discussed fully by K. Terzaghi,[3] Hon. M. ASCE. Even with the introduction of more elaborate methods, which take into account the flexibility of the pile, the classical methods give the preliminary design which is then modified in accordance with the pile flexibility effect. P. W. Rowe's method[4] considers the influence of pile flexibility in modifying the bending moment established by

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SM 1, February, 1960.

1 Prof. of Civ. Eng., Univ. of Fla., Gainesville, Fla.

2 "Design of Sheet-Piling Bulkheads," by R. A. Pennoyer, Civil Engineering, 1933 November, p. 615.

3 "Theoretical Soil Mechanics," by K. Terzaghi, J. Wiley & Sons, 1943.

4 "Anchored Sheet-Pile Walls," by P. W. Rowe, Proceedings Inst. of C. E., London, 1953 Vol. 1, Part 1, pp. 27-70.

the free earth support method, and Terzaghi has discussed[5] the utilization of Rowe's method in practical design.

In the present methods for calculating the loads and moments resulting from earth pressures, the free earth support method involves the solution of a cubic algebraic equation, and the elastic line method utilizes the methods of graphic statics for moments and deflections. Both types of solutions are straightforward but time consuming. The author has found that the numerical method developed[6] by N. M. Newmark, F. ASCE, provides a simple and rapid solution by each of the three classical bulkhead design methods. The use of numerical method for this type of problem has been mentioned previously,[7] but the essential details of the computations were not given. It is the purpose of this paper to describe briefly the principles involved in simplifying these bulkhead design methods.

In the examples, the bulkhead has been rotated into a horizontal position to simplify the arrangement of the computations. The upper free end of the bulkhead is placed at the left margin of the sheet. For convenience of reference, Appendix A contains an elementary description of the Newmark procedure as it applies to cantilever beams.

CLASSICAL DESIGN ASSUMPTIONS

The anchored bulkhead is held in equilibrium by the loads of the soil retained by the bulkhead, and by resistances provided by the anchor and the soil pressure on the embedded length of the bulkhead. Soil pressures are determined as active or passive Coulomb pressures depending upon whether the bulkhead is moving away from or into the soil mass. Fig. 1 illustrates the limiting pressure distributions on the bulkhead when the embedded length is considered (a) to receive "free earth support," and (b) to receive "fixed earth support." The bending moment in the pile and the length of embedment required depend upon the assumption regarding the type of earth support at the lower end of the pile.

Design Based on Free Earth Support.— Free earth support represents the limiting resistance reached when the entire embedded portion of the bulkhead moves outward. Horizontal soil loads are developed on both the loading and restraining surfaces of the bulkhead because of the bulkhead movement and the overburden weight, as shown in Fig. 2(a). If the bulkhead is considered to be fixed at a particular depth, z, the resultant forces produced by the soil loads above this depth may be designated by P_P for the passive force and by P_A for the active earth pressure force. The point of application of each of these forces is at z_P and z_A, respectively, as shown on Fig. 2(b). Stability considerations require that the summation of the moments of these forces about the anchor point be zero, or

$$M_{A_P} = P_A z_A - P_P z_P = 0 \quad \ldots\ldots\ldots\ldots \quad (1)$$

---

5 "Anchored Bulkheads," by K. Terzaghi, Trans. ASCE, 1954, Vol. 119, pp. 1243-1280, disc. 1281-1324.

6 "Numerical Procedure for Computing Deflections, Moments, and Buckling Loads," by N. M. Newmark, Trans. ASCE, 1943 Vol. 108, pp. 1161-1188, disc. 1189-1234.

7 "Analysis for Sheet-Pile Retaining Walls," by F. E. Richart, Jr., Trans. ASCE, 1957 Vol. 122, pp. 1113-1132, disc. 1133-1138.

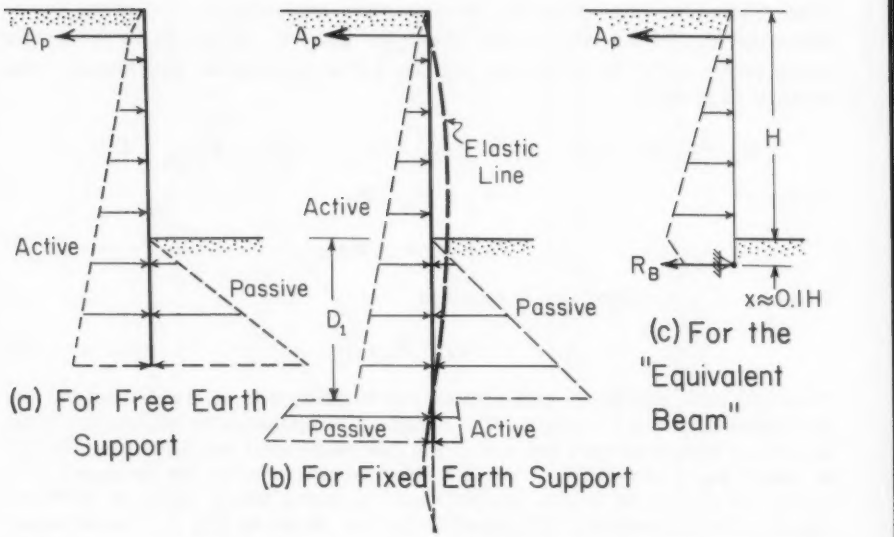
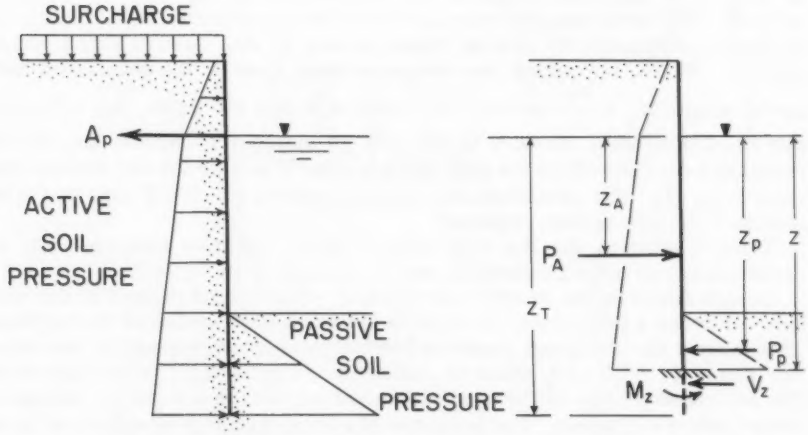


FIG. 1.—LIMITING SOIL PRESSURE DISTRIBUTIONS ON THE ANCHORED BULKHEAD



(a) Soil and Anchor Loads on Bulkhead. (b) Resultant Forces at Depth z Due To Soil Loads Only.

FIG. 2.—BULKHEAD WITH FREE EARTH SUPPORT

As explained in Appendix I, the Newmark procedure permits a rapid evaluation of the total shear V_z, above a given evaluation z, and of the total moment, M_z, of the soil forces about the point z. If we ignore the anchor force temporarily, as shown on Fig. 2b, the moment of the soil forces at the depth z is given by

$$M_z = P_A(z - z_A) - P_p(z - z_p) = (P_A - P_p)z - P_A z_A + P_p z_p$$

or since

$$V_z = P_A - P_p,$$

$$M_z = V_z z - M_{Ap} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

When Eq. 1 is satisfied, Eq. 2 becomes

$$M_z = V_z z \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

Thus the total soil loads and moments in a cantilever pile are required for each interval along the length. The computation proceeds by starting from the upper free end of the pile and continuing downward until the depth z is reached at which Eq. 3 is satisfied. Table 1 illustrates the use of the Newmark procedure to determine loads, shears, and moments which exist at different depths along a particular bulkhead loaded as shown in Fig. 3. These values are given in lines 1 through 6 of Table 1. Line 11 is the value of shear V_z from line 4 multiplied by the corresponding depth z below the anchor point. The depth at which the value of moment, M_z in line 6 equals the value of V_z z in line 11 is the depth required for equilibrium by the free earth support method. This point can be estimated from the numbers in Table 1, or it can be found graphically be plotting these values, as was done in Fig. 4. At the depth z = 46.2 ft, the anchor load is numerically equal to the unbalanced horizontal shear ($V_z = \frac{M_z}{z}$) or for the example it has the value, A_p = 29,700 lb per ft. The bending moment in the pile is obtained by adding the moment given by z A_p (line 12) to the soil moment (line 6) to give the net bending moment (line 13). The maximum net bending moment of -186 ft-kips per ft determines the pile section required.

Thus, a solution for the free earth support condition requires only the computations on lines 1 through 6, and 11 through 13 in Table 1.

Design Based on the Elastic Line Method.—Fixed earth support of the bulkhead provides a restraining moment on the embedded portion of the bulkhead. This causes the deflected shape of the bulkhead to correspond to the dashed line shown in Fig. 1(b), which is called the "elastic line" of the structure. The embedded portion of the bulkhead moves outward to a depth D_1 and moves inward below this depth. For purposes of computation,[8] it is sufficient to establish the depth D_1 and to compute the total required depth D as 1.2 D_1.

The elastic line method is based on the assumptions that (a) at the depth D_1, below the dredge line, the bending moment is zero, the slope of the pile is zero (vertical) and the outward deflection is zero, and (b) that the outward deflection of the anchor point is zero. The slope of the pile at depth D_1 will not actually be zero, but it will be so small that assuming it to be zero causes

8 "Theoretical Soil Mechanics," by K. Terzaghi, J. Wiley & Sons, 1943, p. 225.

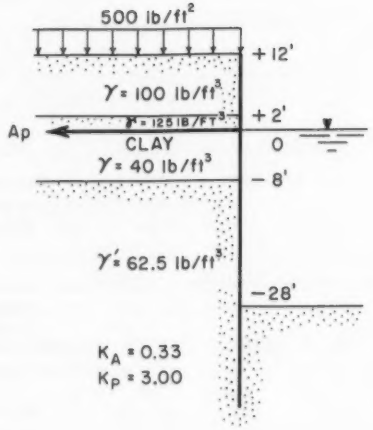


FIG. 3.—CONDITIONS FOR THE EXAMPLE BULKHEAD

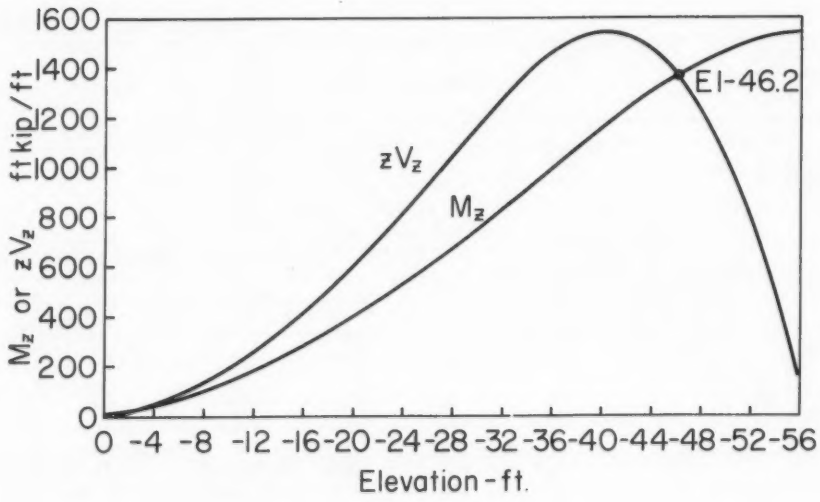


FIG. 4.—GRAPHICAL SOLUTION FOR REQUIRED DEPTH OF EMBEDMENT OF BULKHEAD BY THE "FREE EARTH SUPPORT" METHOD

TABLE I - COMPUTATION OF MOMENTS AND DEFLECTIONS FOR BULKHEAD LOADED AS SHOWN IN Fig. 3

Loads, Shears, Moments, and Deflections Due To The Soil Loads Only

Line	Elevation	+12′	+8′	+4′	0	-4′	-8′	-12′	-16′	-20′	Common Terms	Units
(2)	Soil Pressure p	165	297	429	578	1750	1910	2070	683	766	848	931		lb/ft^2
(3)	Eq.Conc.Force $\bar{p}$	627	1782	2591	1585	5410	11460	6050	2132	4595	5089	5585	h/6	lb/ft
(4)	Shear V_z	627	2409	5000		11995	23455		31637	36232	41321		h/6	lb/ft
(5)	Moment M_z	0	627	3036	8036	20031	43486	75123	111355	152676	h^2/6	ft lb/ft
(6)	Moment M_z	0	1.67	8.10	21.4	53.4	116	200	297	407		ft kip/ft
(7)	Angle Change α	0	-1.67	-8.10	-21.4	-53.4	-116	-200	-297	-407	1/EI	1/ft
(8)	Conc.Ang.Ch. $\bar{\alpha}$		-0.0248	-0.1041	-0.2755	-0.6714	-1.413	-2.413	-3.577	-4.899	1000h/12 EI	
(9)	Slope θ				(0)	-0.276	-0.947	-2.360	-4.773	-8.350	1000h/12 EI	
(10)	Deflection y				0	-0.28	-1.22	-3.58	-8.36	-16.7	1000h^3/12 EI	ft
	For Free Earth Support Solution											
(11)	$\Sigma \cdot V_z$					16.0	93.8	210	338	496		ft kip/ft
(12)	$\Sigma A_p = M_h$ (Free Earth Support)				0	-119	-237	-356	-474	-593		ft kip/ft
(13)	Net Moment = Line(6) + Line(12)				21.4	-65.6	-121	-156	-177	-186		ft kip/ft
	For Fixed Earth Support Solution											
(14)	$\Sigma A_p = M_h$ (Fixed Earth Support)				0	-110	-220	-330	-440	-550		ft kip/ft
(15)	Net Moment = Line(6) + Line(14)				21.4	-56.6	-104	-130	-143	-143		ft kip/ft
	For Equivalent Beam Solution											
(16)	$\Sigma A_p = M_h$ (Equivalent Beam)				0	-104	-207	-311	-415	-519		ft kip/ft
(17)	Net Moment = Line(6) + Line(16)				21.4	-50.6	-91.0	-111	-118	-112		ft kip/ft

TABLE I - COMPUTATION OF MOMENTS AND DEFLECTIONS FOR BULKHEAD LOADED AS SHOWN IN Fig. 3 (con't)

Line	Loads, Shears, Moments, and Deflections Due To The Soil Loads Only													
(1)		-24′	-28′	-32′	-36′	-40′	-44′	-48′	-52′	-56′	-60′	-64′	Common Terms	Units
(2)		1013	1096	429	-239	-907	-1574	-2242	-2909	-3577	-4244	-4912		lb/ft^2
(3)		6079	5826	1954 | 619	-1424	-5439	-9444	-13450	-17454	-21460	-25465	-29471	h/6	lb/ft
(4)	46906	52985	58811	61382	59948	54509	45065	31615	14161	-7299	-32764		h/6	lb/ft
(5)		199582	252567	311377	372760	432708	487217	532282	563897	578058	570759	537995	h^2/6	ft lb/ft
(6)		532	674	830	994	1150	1300	1420	1504	1541	1522	1435		ft kip/ft
(7)		-532	-674	-830	-994	-1150	-1300	-1420	-1504	-1541	-1522	-1435	1/EI	1/ft
(8)		-6.401	-8.102	-9.968	-11.920	-6.59 | -7.22	-7.50 | -8.06	-8.30 | -8.71	-8.87 | -9.13	-9.24 | -9.24	-9.20 | -9.00	-8.82 |	1000h/12 EI	
(9)	-13.249	-19.650	-27.750	-37.720	-49.640	-63.430	-79.000	-96.010	-114.01	-132.44	-150.7			1000h/12 EI	
(10)		-30.0	-49.6	-77.4	-115	-165	-228	-307	-403	-517	-650	-801	1000h^2/12 EI	ft
	For Free Earth Support													
(11)	688	918	1176	1391	1519	1526	1382	1054	510	-282	-1354			ft kip/ft
(12)		-712	-830	-949	-1067	-1186	-1305	-1423						ft kip/ft
(13)		-180	-156	-119	-73	-36	-5	+3						ft kip/ft
	For Fixed Earth Support													
(14)		-660	-770	-880	-990	-1100	-1210	-1320	-1430	-1540				ft kip/ft
(15)		-128	-96	-50	+4	+50	+90	+100	+74	+1				ft kip/ft
	For Equivalent Beam Method													
(16)		-622	-726	-830										ft kip/ft
(17)		-90	-52	0										

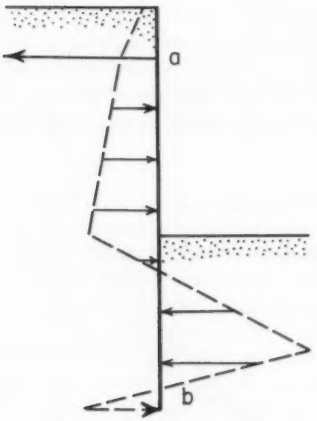
insignificant errors. This is a statically indeterminate problem which is solved by making use of the deflection condition that the anchor point has zero lateral displacement with respect to a point on the pile at the embedded depth D_1.

In the process of determining the required depth of embedment, the bending moments produced in a cantilever beam by the soil loads alone are computed to a depth slightly below the anticipated depth D_1. These moments are given in Table 1 to a depth of El. - 64 ft for the bulkhead shown in Fig. 3. Then the outward deflection of the anchor point, caused by the soil loads alone, can be determined by utilizing the moment-area method of structural mechanics. Figs. 5(a) and (b) show the soil loads and bending moments, respectively, acting on the bulkhead with fixed earth support. Fig. 5(c) illustrates the outward movement of the anchor point which would occur as a result of these soil loads if the anchor restraint is removed and the bulkhead acts as a cantilever fixed at depth z. The first proposition of the moment-area method states that the change in slope between any two points in a beam is equal to the area under the M/EI curve between the two points. The second proposition states that the deflection of any point in a beam measured from the tangent to the elastic curve at a second point, is numerically equal to the moment of the area under the M/EI curve between the two points, about the point for which the deflection is desired. Thus, by using these propositions, the change in slope between points a' and b in Fig. 5(c) may be evaluated as the angle θ. Also the deflection y_a at the anchor point due to the soil loads may be determined. This value of deflection due to the soil loads must be counterbalanced by an equal and opposite deflection provided by the anchor pull.

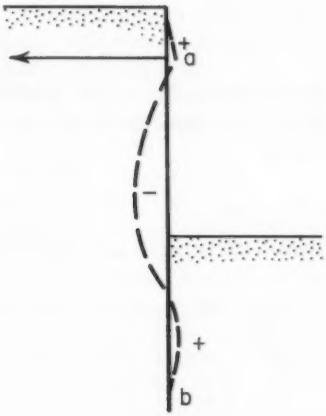
In the conventional graphical procedure for determining the required depth of embedment, a depth z (corresponding to an embedded depth D_1) is guessed from inspection of the M_z diagram (shown in Fig. 4), and the bulkhead is considered to be fixed at that elevation. The outward deflection of the anchor point due to the soil loads is evaluated graphically and is compared to the inward deflection which results from the concentrated anchor load of value, $A_p = \frac{M_z}{z}$ applied to the cantilever structure. If these deflections are not approximately of equal magnitude, the process must be repeated. Usually several trials are required.

By use of the Newmark procedure, the deflection of the bulkhead due to soil loads is most conveniently calculated by using the anchor point as a reference and working downward toward the pile tip. By considering the bulkhead to be vertical (slope = 0) and to have no outward deflection at the anchor point, the deflection y_z at any depth can be computed directly as shown in Table 1. The resulting deflection curve is illustrated by Fig. 5(d). Since the change in slope between any two points along the length a' b is established by the area of the M/EI diagram between these two points, it is evident that the same elastic curve exists between a' and b in Figs. 5(c) and (d). The curve of Fig. 5(d) may be obtained by rotating the curve of Fig. 5(c) through an angle of θ. Thus, to determine the anchor point deflection, y_a, by means of the numerical procedure which defines y_z and θ_z at successive depths we can use the relation

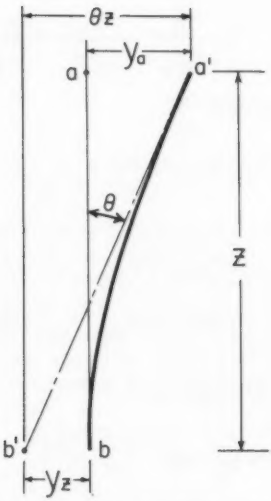
$$y_a = \theta_z z - y_z \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$



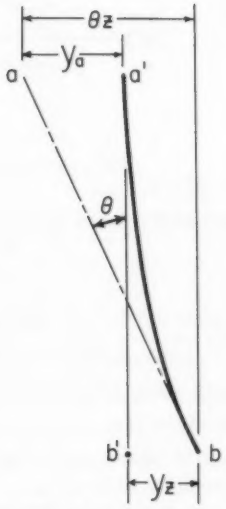
(a) Soil Pressures on Bulkhead



(b) Bending Moment in Bulkhead due to Soil Loads and Anchor Force



(c) Deflection of Bulkhead due to Soil Loads Only (Anchor Removed)



(d) Deflection of Bulkhead due to Soil Loads Only – Considering Fixity at the Anchor Point

FIG. 5.—BULKHEAD WITH FIXED EARTH SUPPORT

This deflection must be equal to that produced by the anchor pull, or

$$y_{Ap} = \frac{Ap\ z^3}{3\ E\ I} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

The anchor pull, A_p, must produce a moment at depth z which is equal in magnitude and opposite in sense to the soil load moment M_z from line 6 of Table 1, or

$$Ap = \frac{M_z}{z} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

After substituting Eq. 6 into Eq. 5 and setting Eqs. 4 and 5 equal to each other,

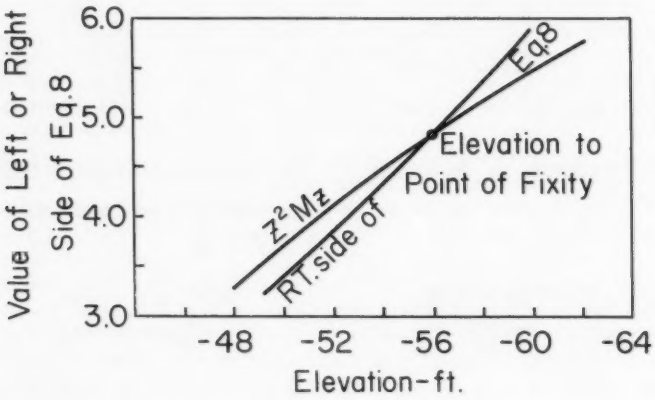


FIG. 6.—GRAPHICAL SOLUTION OF EQ. 8

the final expression to be satisfied is:

$$\frac{M_z\ z^2}{3\ E\ I} = \theta_z\ z - y_z \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

At a particular depth, z_1, the numerical values of M_{z_1}, θ_{z_1}, and y_{z_1}, will satisfy Eq. 7, which establishes that

(a) the bending moment in the pile is zero at depth z_1,
(b) the slope of the pile is zero at depth z_1
(c) the deflection of the pile is zero at depth z_1; and
(d) the anchor point deflection is zero.

In the actual evaluation of Eq. 7, by substituting values from Table 1, the EI-terms drop out and the expression reduces from

to

$$\frac{M_z\ z^2}{3\ E\ I} = (\text{No. for } \theta_z)\ \frac{1000\ h\ z}{12\ E\ I} - (\text{No. for } y_z)\ \frac{1000\ h^2}{12\ E\ I}$$

$$z^2\ M_z = (\text{No. for } \theta_z) 250\ h\ z - (\text{No. for } y_z) 250\ h^2 \ldots\ldots \quad (8)$$

which may be solved graphically as indicated on Fig. 6. The intermediate computations are given in Table 2. Note that for use in Col. 3 of Table 2, it is necessary to evaluate the slope, θ_z, and the deflection at the same z depth. The details of this computation are explained in Appendix 1.

The graphical solution of Eq. 8 on Fig. 6 determines the point of fixity at El. - 56 ft, or D_1 = 28 ft. Then the total depth of embedment

$$D = 1.2\ D_1 = 33.6 \text{ ft}$$

The anchor pull is $A_p = M - 56/56 = 27{,}500$ lb per ft. The net bending moment is determined by adding line 14 of Table 1 to line 6 to give line 15, which determines a maximum moment of -143 kips per ft.

Use of the Equivalent Beam Method.—The equivalent beam is a simplification of the elastic line method obtained by considering an imaginery hinge in the bulkhead. The location of this zero moment point depends upon the characteristics of the backfill and the material supporting the embedded bulkhead length. For sandy materials supporting the bulkhead and in the backfill, the zero moment point is approximately at 0.1 H below the dredge line, where H

TABLE 2.—EVALUATION OF QUANTITIES IN EQ. 8

z, in feet (1)	$\frac{z^2 M_z}{10^6}$ (2)	(No. for θ_z) $\frac{1{,}000\ z}{10^6}$ (3)	(No. for y_z) $\frac{4{,}000}{10^6}$ (4)	Col. (3) - Col. (4) 10^6 (5)
48	3.272	4.190	1.228	2.962
52	4.067	5.454	1.612	3.842
56	4.833	6.899	2.068	4.831
60	5.479	8.498	2.600	5.898

is the distance from the top of the pile to the dredge line as shown in Fig. 1(c). For the example considered herein, H = 40 ft, and the hinge is assumed to be located 0.1 H = 4 ft below the dredge line, or at El. - 32 ft.

The moment given in line 6 of Table 1 is due to the soil loads alone acting on a cantilever pile. In order to produce a point of zero moment at El. - 32 ft, an anchor pull must be applied which produces a moment equal and opposite to the soil moment at that elevation. For the equivalent beam the anchor pull must be

$$A_p = \frac{M_z(\text{at El-32 ft})}{32 \text{ ft}} = \frac{830}{32} = 26.0 \text{ kips per ft or } 26{,}000 \text{ lb per ft}$$

The distribution of anchor pull moment with depth is given in line 16, Table 1 and when this is added to line 6 the net moment is obtained. The net moment is given in line 17. By the equivalent beam method the maximum bending moment in the bulkhead amounts to - 118 kips per ft.

The concentrated reaction R_B at the hinge location (Fig. 1(c)) is obtained by subtracting the anchor pull from the total soil shear load given in line 4 of Table 1:

$$R_B = 60{,}765 \frac{h}{6} - A_p = 40{,}500 - 26{,}000 = 14{,}500 \text{ lb per ft} \quad . \quad . \quad . \quad (9)$$

From Terzaghi,[9] the expression for the total depth of embedment required is

$$D = 1.2\,D_1 = 1.2\left[\frac{3}{2}H\frac{K_A}{(K_P - K_A)} - \frac{x}{2} + \sqrt{\frac{6\,R_B}{(K_P - K_A)\gamma}}\right] \quad \ldots \quad (10)$$

For this example, $x = 0.1\,H = 4$ ft, $H = 40$ ft, $K_P = 3.0$, $K_A = 0.33$, and $\gamma = 62.5$ lb per ft. After substituting these values in Eq. 10, D was found to be 33.9 ft.

CONCLUSIONS

Design quantities can be determined rapidly by each of the three methods (Table 3) after the soil loads and moments have been determined for a cantilever bulkhead. The use of Newmark's numerical method provides a rapid means for determining the bending moments and deflections of the bulkhead due to the soil loads. By use of the methods described in this paper, the total time required to determine the design quantities for a particular bulkhead installation is reduced considerably.

TABLE 3.—SUMMARY OF DESIGN VALUES FOR EXAMPLE

Method	Required embedment depth, D, in feet	Anchor pull, A_p, in pounds per foot	Maximum bending moment, M, in foot-pounds per foot
(1)	(2)	(3)	(4)
Free Earth Support	18.2	29,700	186,000
Elastic Line	33.6	27,500	143,000
Equivalent Beam	33.9	26,000	118,000

ACKNOWLEDGMENT

The use of Newmark's method for solving the classical bulkhead design problems was developed by the author during his work with Moran, Proctor, Mueser, and Rutledge, Consulting Engineers, in the summer of 1953. This firm has found the method convenient and has adopted its use for the study of bulkhead problems.

W. J. Grantham, Jr., Graduate Asst. in Civ. Engrg. at the Univ. of Florida, recently pointed out simplifications for presenting the free earth support and elastic line methods. These contributions have been incorporated in the text and the author wishes to give special acknowledgment to Mr. Grantham for his improvements in the method. The author also wishes to acknowledge the

9 "Theoretical Soil Mechanics," by K. Terzaghi, J. Wiley & Sons, 1943, p. 227.

helpful comments and criticisms contributed by W. H. Zimpfer, Assoc. Prof. of Civ. Engrg., Univ. of Florida.

APPENDIX I

The following is a brief description of the numerical procedure for analyzing beam problems which was developed by Newmark.[6] The advantage of this procedure is that almost any beam problem can be reduced rather easily to a problem of simple arithmetic. Fig. 7 shows a cantilever beam carrying a load which has a random distribution of intensity. If we cut the load at various intervals along the span, and apply the load within each interval to a simple supported "auxiliary" beam, the reactions required to support each end of

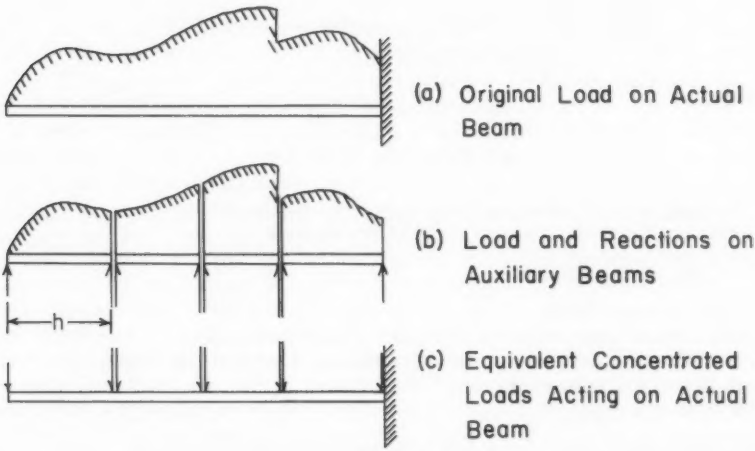

(a) Original Load on Actual Beam
(b) Load and Reactions on Auxiliary Beams
h
(c) Equivalent Concentrated Loads Acting on Actual Beam

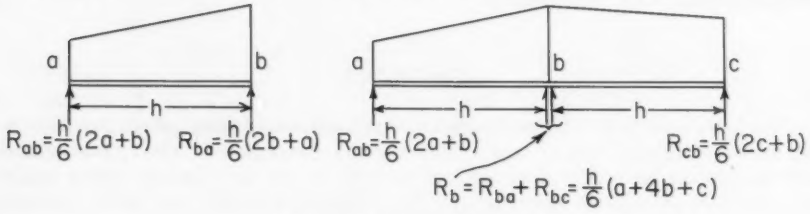
FIG. 7.—METHOD OF REPLACING A DISTRIBUTED LOAD BY CONCENTRATED LOADS ON A BEAM

the simple beam can be computed. If the length of the simple beam is designated by the term, h, it is obvious that if h is very small, a simple trapezoidal load distribution will be a good approximation for the actual distribution over this interval. As the length h increases, the trapezoidal loading will probably be more inaccurate. In order to make a better approximation to a curved loading diagram, a parabola may be passed through any three given points.

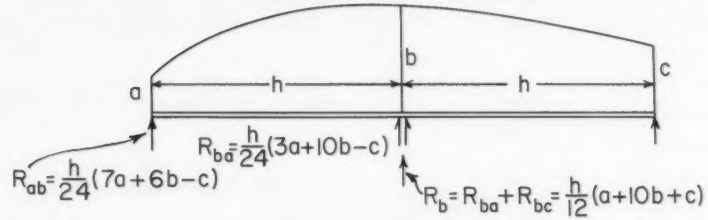
Fig. 8 gives the expressions for the reactions at the ends of the auxiliary beams which carry trapezoidal or parabolic loads. For a given problem either set of reactions may be used and the results will be nearly identical if h is taken very small. For large values of h, the straight or curved loading diagram will be chosen, whichever fits the actual conditions the better.

After the end reactions for the auxiliary beams have been computed, these reactions are now applied to the actual beam. We have replaced the distributed load on the beam by a set of "concentrated" loads which are statically equivalent to the original load. This condition is illustrated in Fig. 7(c).

The sign convention described in Newmark's paper will be followed in this



(a) Trapezoidal Loading



(b) Parabolic Loading

FIG. 8.—EXPRESSIONS FOR THE REACTIONS AT THE ENDS OF THE AUXILIARY BEAMS

discussion. It is as follows:

+ loads act upwards;
+ shears act upward on the left side of a cut section;
+ moments produce compression in the top fibers of the beam;
+ deflection is downward; and
+ x is to the right.

The deflections and slopes are assumed to be small so that

$$\frac{d^2y}{dx^2} \approx \frac{\frac{d^2y}{dx^2}}{\left[1+\left(\frac{dy}{dx}\right)^2\right]^{\frac{3}{2}}} = -\frac{M}{EI}$$

and the ordinary beam relations hold, that is

y = deflections

$\frac{dy}{dx} = \theta$ = slope

$\frac{d^2y}{dx^2} = \frac{M}{EI} = \alpha$ = angle change

$\frac{d^3y}{dx^3} = -\frac{dM}{dx}\frac{1}{EI} = -\frac{V}{EI}$ or $EI\frac{d^3y}{dx^3} = V$ = shear

$-EI\frac{d^4y}{dx^4} = \frac{d^2M}{dx^2} = \frac{dV}{dx} = p$ = distributed load

To determine the shear at any point in the beam, it is only necessary to start at some point of known shear (usually a point of zero shear), and add up the concentrated loads as the computation progresses across the beam. Fig. 9 is a sample problem consisting of a cantilever beam carrying a uniform load of 10 lb per ft. The equivalent concentrated loads are determined by use of the equations in Fig. 8(a) (since the loading diagram is a straight line) and are designated as $\bar{p}$ is Step. No. 2. To find the shear at any point, start from the left end (free end) of the beam where the shear is zero, and add the values of the equivalent concentrated loads as the computation proceeds toward the right end. The shear diagram consists of a series of steps as shown in Fig. 9(b). If the value of the shear in each auxiliary beam is superposed on the step-shear diagram, the exact diagram is obtained. For our purpose, however, it is sufficient to note that the step shear diagram coincides with exact value at mid-span of each h-interval. Consequently, in Step 3, the value of shear is written in the middle of the h-interval. When it becomes necessary to evaluate the shear at the end of the h-interval, the concentrated load acting at the right end of the h-length must be computed and added to the value of the average shear over this h-length. That is, at a point b, as shown on Fig. 8, R_{ba} must be computed and added to the value of average shear over the length ab to give the correct shear at point b.

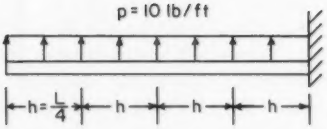
In order to evaluate the moments, it is necessary to multiply the value of shear by its moment arm to the point in question. Ordinarily it is easier to solve these problems by assuming all values of h to be equal. Then to find moment, the shears are simply added from left to right and the common multiplication factor is carried as a common term in the right hand margin of the problem. In Fig. 9(a) h/6 is the common term for the concentrated loads and shear. The moment requires multiplication by another h so the common term now becomes $h^2/6$. The value of moment is exact at each h point.

The distributed angle change, $\alpha = M/EI$, constitutes the load on the conjugate beam. Since the moment diagram is curved the concentrated reactions are computed from Fig. 8(b). The term h/12 becomes a factor in the common term. The concentrated angle changes correspond to the concentrated loads of Step 2. For Step 7, to determine the slope (or the shear on the conjugate beam), start from the right end of the beam. When proceeding from the right end of the beam to the left, subtract the next number. Thus for zero slope at the right end, the concentrated angle change is -2430, so the slope at mid-h is +2430, etc., until the left end is reached, where the slope is $+7680 \frac{h^3}{72 E I}$. The slope at this point is consistent with the sign convention mentioned previously.

For deflections, again start at the right (fixed) end and subtract the values of slope as computation proceeds toward the left end. The value of deflection at the left (free) end is found to be $-23{,}000 \frac{h^4}{72 E I}$, or $y = -\frac{p L^4}{8 E I}$.

This answer is identical to the exact answer because the loading diagrams p, and α, consist of straight lines and parabolic curves, respectively. Thus the expressions given in Fig. 8 gave exact answers instead of approximations.

The foregoing procedure applies to any beam problems for which the load distribution can be established. The method is not complicated by a change of moment of inertia of the beam along the span as might occur when reinforced pile sections are used. This simply enters the picture in Step 5 (Fig. 9) where the value of moment is divided by the moment of inertia to give the ordinate to the angle change diagram. The reader is referred to Newmark's paper for more details on this method.



STEP NO							COMMON TERMS	UNITS
1. DISTRIBUTED LOAD p	10	10	10	10	10			$\frac{lb}{ft}$
2. EQUIVALENT CONCENTRATED LOAD $\overline{p}$	30	60	60	60	30		$\frac{h}{6}$	lb
3. SHEAR V	(0)	30	90	150	210	(240)	$\frac{h}{6}$	lb
4. MOMENT M	0	30	120	270	480		$\frac{h^2}{6}$	ft lb
5. ANGLE CHANGE $\alpha = M/EI$	0	−30	−120	−270	−480		$\frac{h^2}{6EI}$	$\frac{I}{ft}$
6. CONCENTRATED ANGLE CHANGE $\overline{\alpha}$	−30	−420	−1500	−3300	−2430		$\frac{h}{12}\ \frac{h^2}{6EI}$	
7. SLOPE θ	(7680)	7650	7230	5730	2430	(0)	$\frac{h}{12}\ \frac{h^3}{6EI}$	
8. DEFLECTION y	−23040	−15390	−8160	−2430	0		$\frac{h^2}{12}\ \frac{h^2}{6EI}$	ft

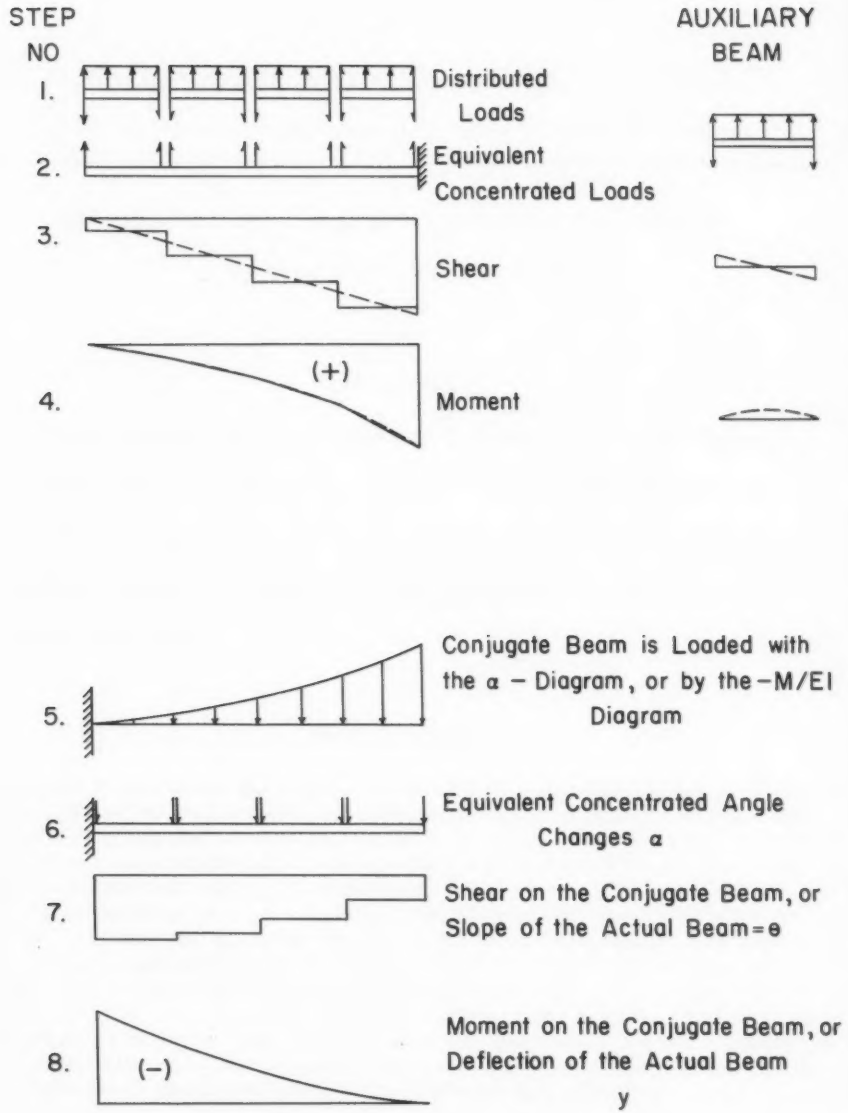
$$y_{max} = \frac{-23040h^4}{72EI} = \frac{-1.25L^4}{EI} = \frac{-pL^4}{8EI} \quad \text{(Upward Deflection)}$$

NOTES: 1. TRAPEZOIDAL LOADING EXPRESSIONS USED TO GET EQUIVALENT CONCENTRATED LOADS FROM DISTRIBUTED LOADS.

2. PARABOLIC LOADING EXPRESSIONS USED TO GET EQUIVALENT CONCENTRATED ANGLE CHANGES FROM DISTRIBUTED ANGLE CHANGES.

(a) COMPUTATION OF THE DEFLECTION OF A CANTILEVER BEAM CARRYING A UNIFORM LOAD.

FIG. 9.—ILLUS-



(b) Diagrams of the Values Computed in (a).

TRATIVE EXAMPLE

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

INSTALLATION AND OPERATION OF DEWATERING SYSTEMS

By David A. Werblin,[1] M. ASCE

SYNOPSIS

The three principal types of dewatering systems—sumps, wellpoints, and deep wells—are described. Conditions under which each type may be used are discussed, equipment and methods of installation are presented, together with methods of coping with some of the problems that arise in the field.

INTRODUCTION

Ground water is the inescapable companion of many construction projects. Excavation of wet soil is inefficient and costly. Unstable, saturated soils cause equipment to bog down and valuable time to be wasted. Nevertheless, economy sometimes requires dealing with the problem of ground water rather than averting it by structure redesign or relocation.

Any dewatering program consists of two distinct phases—design of the dewatering system and executing the plan in the field. Frequently, data used in design computations prove to be different from actual conditions encountered at the site. Unless discovered quickly during installation, such unexpected conditions may turn an apparently simple job into a costly muddle. The purpose of this paper is to describe the installation and operation of sumps, wellpoints, and deep wells—the most commonly-used dewatering systems—and to discuss the methods of coping with some of the problems that arise in the field.

Note.—Discussion open until July 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SM 1, February, 1960.

[1] Vice President, Griffin Wellpoint Corporation, New York, N. Y.

Most excavations can be dewatered by any one of the aforementioned methods. Economy and practicability are the usual criteria for selecting which of the three methods is to be used. Dewatering by pumping from sumps, termed sumping or open-pumping, is the simplest, and in many cases, the most economical method. It is used primarily in gravelly formations which inherently resist sloughing, in certain sheeted excavations, and in excavations of limited size where the quantity of water to be pumped is small. Sumping is also the first method tried when an unexpected ground water problem is encountered. As each sump (or pumping pit) requires a separate pump, open-pumping becomes impractical if more than a few sumps are required. In such cases, either wellpoints or deep wells are used for the dewatering.

Wellpointing, although more elaborate, is still a comparatively simple process. It is usually the method employed in pervious soils where sumping is not practicable. Economically, one advantage of a wellpoint system is the ability to operate a large number of wellpoints from a single pump station.

Deep wells are most effective where the water-bearing soil is clean, uniform, and highly pervious, or where installation problems preclude the use of sumps or wellpoints. Although each deep well requires the installation of a costly well and a separate pump, a few deep wells can sometimes be the more economical method to dewater a site. As with sumps, the economic advantage of a deep well system decreases rapidly as the number of units increases.

INSTALLATION OF DEWATERING SYSTEMS

The first step toward the successful installation of any type of dewatering system is proper planning. Prior to starting the installation, the following information should be in hand:

1. The dimensioned excavation plan and sections of the entire area to be pre-drained, including grade and subgrade elevations.
2. Boring data for the entire working area from ground surface to at least 10 ft below the deepest subgrade.
3. Elevation of the water table, and the expected fluctuation of this level with tides or floods.
4. Location of a hydrant, pond or other source of water for jetting.
5. Location of the disposal point for pump discharge.
6. Excavation and construction program as related to dewatering operations.

Whether sumps, wellpoints, or wells are used, these data provide the basis for field layout and assist in formulating changes when required by unforeseen conditions. Procedures based upon inadequate or inaccurate information can cause unnecessary trouble, and in extremes may lead to serious failures. Careful observation of soil conditions during system installation can reveal potential problems that may be counteracted prior to their actual occurrence.

Sumps.—The pumping equipment required for sumping is relatively simple. The primary item is the sump pump which is usually a self-priming centrifugal pump designed to lift large quantities of muddy water approximately 10 ft to 20 ft. In addition to the sump pump, a suction line and a discharge line are the only other major items of equipment necessary. Although rigid piping may be used, flexible hose is more versatile. An important adjunct to the suction line is a strainer located at the intake end to prevent clogging.

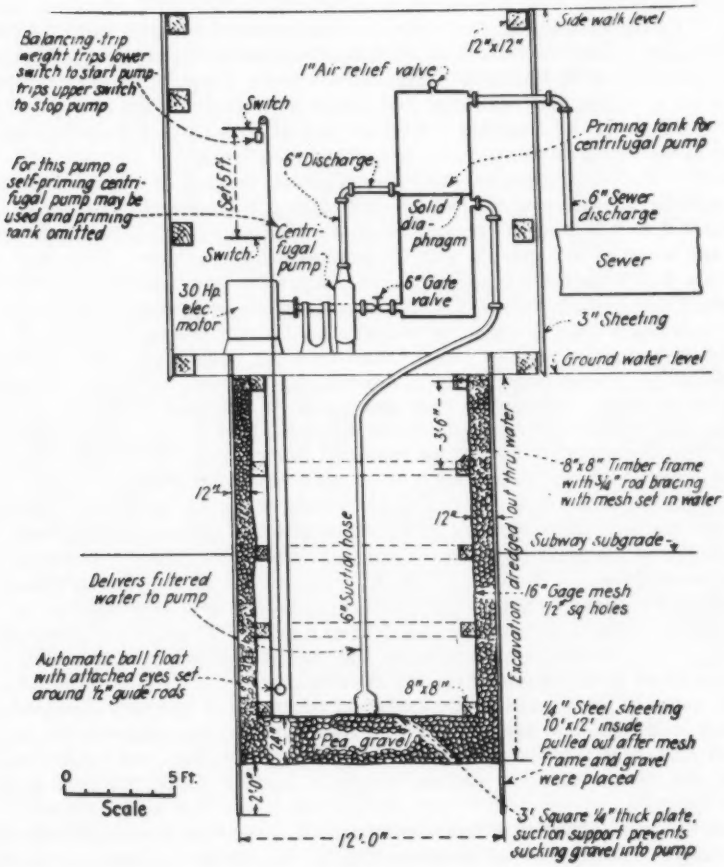

Side walk level
12"x12"
1" Air relief valve
Balancing trip weight trips lower switch to start pump trips upper switch to stop pump
Switch
Priming tank for centrifugal pump
6" Discharge
For this pump a self-priming centrifugal pump may be used and priming tank omitted
Set 5 ft.
6" Sewer discharge
Solid diaphragm
Centrifugal pump
Switch
Sewer
30 Hp elec. motor
6" Gate valve
3" Sheeting
Ground water level
3'6"
8"x8" Timber frame with 3/4" rod bracing with mesh set in water
12"
12"
Subway subgrade
Excavation dredged out thru water
6" Suction hose
Delivers filtered water to pump
16" Gage mesh 1/2" sq holes
Automatic ball float with attached eyes set around 1/2" guide rods
8"x8"
1/4" Steel sheeting 10'x12' inside pulled out after mesh frame and gravel were placed
24"
Pea gravel
0 5 Ft.
Scale
2'0"
12'0"
3' Square 1/4" thick plate, suction support prevents sucking gravel into pump

FIG. 1.—SHEETED SUMP USED IN GENERAL CONSTRUCTION
(COURTESY OF SPENCER, WHITE & PRENTIS, INC.)

For dewatering small trenches and excavations, the intake end of the suction line is laid in a low spot in the excavation. As the size of the excavation increases, pits dug at strategic points within the excavation expedite the collection of water. Where they interfere with the work or structure, these pits are located outside of the excavation and are sheeted to prevent wall collapse.

Shown in Fig. 1 is an elaborate sump used to control the ground water level during the construction of the cross-town Houston Street branch of the Independent Subway in New York City.[2] A pea-gravel blanket, which formed the floor of the sheeted sump, and a gravel wall reduced the possibility of "boils" within the sump and the undermining of the sheeting during pumping. The gravel wall was formed by lining the sump with 1/4-in. mesh screen attached to a framework, filling the resulting 1-ft space with coarse sand and gravel, and then withdrawing the sheeting. During operation, a float-actuated pump kept water in the sump depressed to a predetermined level.

Wellpoints.—The components of a wellpoint system are shown in Fig. 2. The standard dewatering wellpoint essentially is a screened cylinder approximately 2 ft to 4 ft long and from 2 in. to 3 in. in diameter. The screen openings are sized to correspond with the predominant grain size of the soil. This suggests that a variety of standard screen-opening sizes are stocked. Actually a limited number are required to meet practically all average conditions. The most commonly used dewatering wellpoint is the self-jetting type, which is installed by using a stream of water jetted from the lower end. The wellpoint is connected to a collecting manifold, termed header pipe, through a riser pipe and a swing joint. The swing joint has a pair of swinging elbows at each end, a union, and a shut-off valve (clock) for adjusting flow from an individual wellpoint. Where greater flexibility is required rubber swing joints are used. The header pipe, which is connected to one or more wellpoint pumps, is light-weight metal with inlets at 2 ft to 3 ft intervals to receive the swing joints. Pipe sizes vary according to the volume of water to be pumped. Standard header and discharge piping range from 4 in. to 16 in. in diameter; riser and swing joint sizes are from 1-1/2 in. to 2-1/2 in. in diameter. Larger sizes are used occasionally for special conditions. Gate valves are used to isolate pumps and to divide sections of the header line as required. Elbows, tees, gate valves, and lengths of header and discharge line are joined by special steel-with-rubber-sleeve band couplings for rapid assembly.

The heart of the system is the wellpoint pump which is a unit designed specifically to handle both air and water, automatically and continuously, at high suction lifts. Wellpoint pumps, powered by diesel, gasoline, and butane (natural gas) engines or electric motors, range in capacity from 300 gpm to more than 5,000 gpm.

Dewatering to about 18 ft below the water table can be accomplished with a single stage of wellpoints as shown in Fig. 2. Under ideal conditions, and using special high-vacuum equipment, the depth of lowering has been increased to as much as 25 ft.

Dewatering 25 ft or more below the water table requires more than one stage, or tier, of wellpoints. A typical two-stage layout is shown in Fig. 3. Ground is stripped to the level predrained by the first stage, and the next stage installed. Under average conditions, any number of stages can be used, each stage lowering the water level about 15 ft. Operation of the lowest stage of

---

[2] Prentis, E. A. and White, L., 1950, Underpinning Its Practice and Applications. New York: Columbia University Press.

wellpoints may dewater the screens of one or more of the upper stages. When this occurs, the pumps from the upper stages can be removed and used on the lower stages. The remainder of the upper stages—wellpoints, risers, and header line—however, are left in place for removal of the lower stages after job completion("backing out of the excavation").

Dewatering between 18 ft and 25 ft below the water table can be accomplished with a temporary stage of wellpoints eliminating the need for a second permanent stage. The water table is initially lowered by the temporary stage, ground stripped to this level, and a permanent system installed. After the water table has been depressed to the final level, removal of the temporary stage is possible, provided that it will not be needed to remove the permanent system.

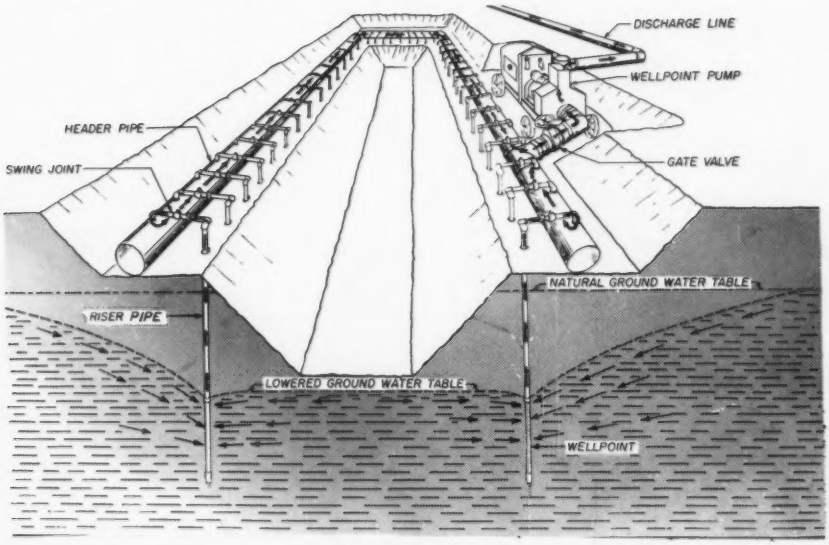


FIG. 2.—COMPONENTS OF A WELLPOINT SYSTEM.

The installation work falls into three categories: setting pumps and discharge piping, laying header lines, and installing wellpoints. The average installation crew consists of 4 to 6 men assisted occasionally by a crane for setting the pumps and spotting the larger sizes of pipe. All connections are accomplished with simple tools, such as pipe wrenches and socket wrenches.

Efficiency in the installation of a wellpoint system begins with the distribution of the equipment. Unloading is planned to eliminate as much rehandling as possible. Pumps are placed initially in their permanent locations, and sections of header pipe are laid out and connected along previously staked-out lines. The header pipe may be laid on the top of the ground where the depth to the water table is less than 15 ft and the required lowering is on the order of a few feet. Otherwise, it should be laid as close to the water table as possible to reduce the suction lift. In all cases, the header should be set back far enough

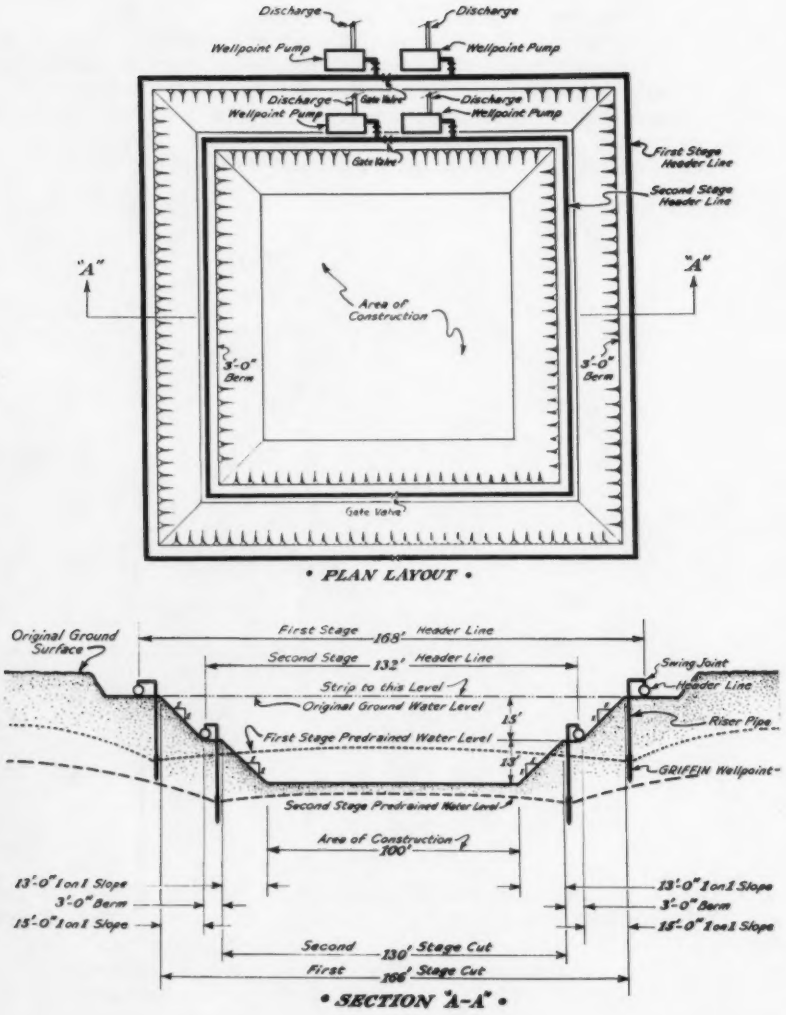


FIG. 3.—TYPICAL TWO-STAGE WELLPOINT SYSTEM.

from the top of slope to be sure that the pipe will not be undermined in the event of sloughing. Wellpoint assemblies are "made up" concurrently and are distributed along the header at design spacing.

Fig. 4 shows schematically the arrangement of a wellpoint ready to be jetted. Actually a crew of 4 or 5 men is used to install the wellpoints as shown in Fig. 5. The required jetting pressure may vary from as little as 25 psi to more than 200 psi depending upon soil formations and jetting hose lengths. Standard jetting pumps have capacities up to about 650 gpm at pressures up to 250 psi. Referring again to Fig. 4, water under pressure is forced through the hose, down the riser pipe and through the inside of the wellpoint forming a jet stream at the wellpoint nozzle. The jet stream breaks up the soil, washes the displaced material to ground surface and allows the wellpoint to sink to the desired depth. To prevent the leakage of air into the system while dewatering to the required depth, the tops of all wellpoint screens are placed at least 2 ft below subgrade. The installation is fast and on the average job approximately fifty wellpoints can be jetted by one crew in a normal working day. Under ideal conditions as many as one hundred wellpoints have been installed in clean sand during an 8-hr day.

After the wellpoint is in place, part of the jetting crew connects the wellpoint to the header by joining the two halves of the swing joint. At the same time the remainder of the crew readies the next wellpoint for jetting.

Soil characteristics, adjacent structures, or limitations of excavating equipment sometimes necessitate the use of sheet piling or H-beams and lagging. In such cases, wellpoints can be located either inside or outside of the sheeting. Placement outside of the sheeting is preferred as the possibility of damage to the wellpoint system is reduced, obstructions in the working space are eliminated, and the installation and removal of the system is facilitated. Where system placement must be made inside of steel-sheeted excavations, risers are recessed in the webs of the sheet piling, if possible, and headers are supported on the wales or on brackets welded to the sheeting. Where system placement must be made inside of excavations sheeted with H-beams and lagging, risers should be recessed in the space between the H-beams, where possible, and headers supported on brackets welded to the H-beams.

Recently the use of ejector pumps has been introduced in dewatering work. A dewatering system using this type of pump ("ejector pump dewatering system") is a modification of the wellpoint dewatering system, and is particularly successful in areas where it is inconvenient or impossible to install a multistage wellpoint system. The primary advantage of an ejector pump dewatering system is that it can be used to operate a multitude of wellpoints in a single stage to depths of 60 ft or more. On the other hand, conventional wellpoint dewatering systems require additional stages when the depth of lowering the water exceeds about 18 ft.

Each ejector pump resembles a wellpoint assembly, having a screen, riser, and swing joint. Ejector pumps are installed using jetting and casing techniques similar to those used for installing conventional wellpoints in difficult terranes (see "Installation Problems"). The ejector pump (Fig. 6) consists of a simply constructed nozzle arrangement, called an ejector or eductor, positioned in the ground at the bottom of two small-diameter riser pipes. A screen is attached just below the eductor. Water from a high pressure manifold flows through one of the pipes to the eductor. Passage of this water through a tapered constriction within the eductor creates a lower-than-atmospheric pressure inducing ground-water flow from the surrounding water-bearing sands

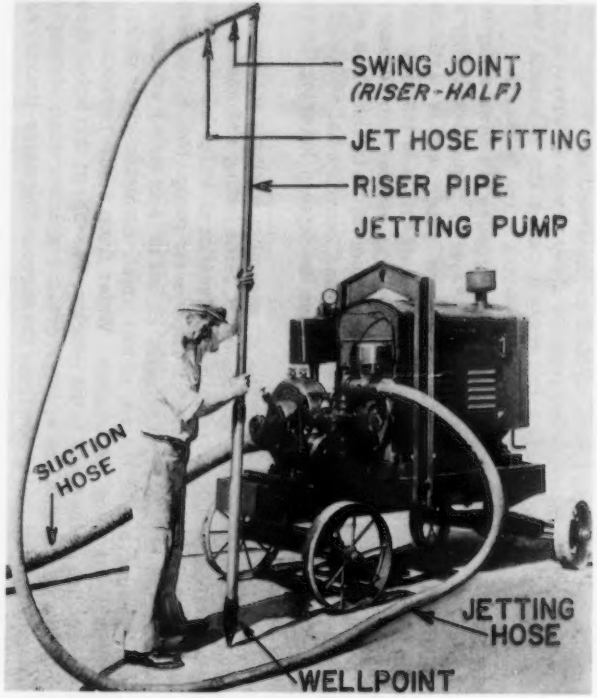


FIG. 4.—WELLPOINT JETTING SET-UP.

FIG. 5.—JETTING A WELLPOINT IN THE FIELD.

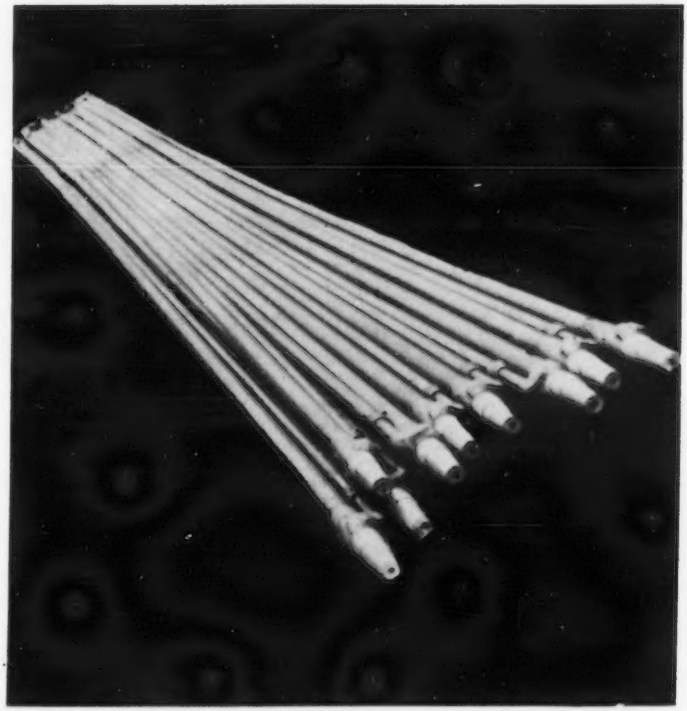


FIG. 6.—TWO-PIPE EJECTOR PUMPS.

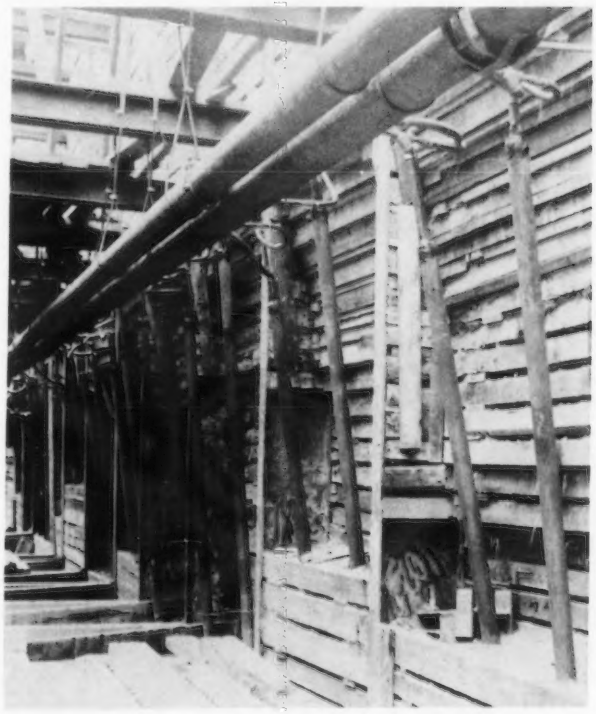


FIG. 7.—EJECTOR PUMP INSTALLATION USED DURING THE CONSTRUCTION OF THE CHRYSTIE STREET SUBWAY.

into the screen. The other pipe discharges the ground water together with the water that enters from the high pressure side of the eductor to a collecting manifold. Part of the water in the collecting manifold is recirculated to the pressure manifold by means of a centrifugal pump; the remainder is discharged to waste.

Commercially, ejector pumps are designated as two types: two-pipe and one-pipe. Actually, both types utilize two pipes and operate on exactly the same principle. In the two-pipe ejector, the two pipes are arranged parallel to and alongside each other. In the one-pipe ejector, one pipe is centered inside the other. Thus, the inside pipe serves as one conduit and the annular space between the two pipes acts as the other conduit.

Shown in Fig. 7 is a segment of the system used to control ground water during the construction of the Chrystie Street Subway, in New York, N. Y., the first large-scale application of ejector pumps for dewatering operations. Detailed elsewhere,[3] more than 700 ejector pump assemblies were operated as a single-stage dewatering unit on this project to lower the ground water 40 ft.

Deep Wells.—Although deep wells and wellpoints utilize the same principle in lowering the ground water, individual wells handle much larger quantities of water than individual wellpoints. Compared to wellpoints, deep wells are installed differently and pumped differently.

One of the outstanding differences is illustrated by the fact that when one well fails, a major part of the dewatering capacity is lost. Whereas, in a wellpoint system, dozens or hundreds of companion wellpoints are always in operation and should a few wellpoints become inoperative, only a small fraction of the capacity is lost.

A deep well system is comprised of three components, namely—the well itself, the pump, and the discharge piping. The well consists of two parts—an open area through which ground water can flow, and a cased section which conducts this water to ground surface. In rock wells, seepage takes place through the walls of the uncased hole. In granular soils, on the other hand, slotted metal screens or perforated casings are required to prevent the walls of the hole from sloughing. Occasionally, wells in decayed rock or in rock containing pockets of loose soil require the use of screens for successful operation.

Three principal types of well-drilling procedures are used for the installation of large-capacity wells; the cable-tool method, the rotary method, and the reverse-rotary method. The selection of the method usually depends upon the magnitude of the job and the type of equipment available locally when the well is needed. In the cable-tool method, the drilled hole is kept open during installation with a steel casing that forms a permanent part of the finished well. On the other hand, the rotary methods require surface casing only, using a fluid medium to sustain the drill hole as described later in this paper. The rotary methods are faster, but involve considerable equipment mobilization costs. Cable-tool rigs are simpler to set up and are more readily available.

A cable-tool rig is shown in Fig. 8. A casing is driven using cross-pieces clamped to the top of a large steel bit. The entrapped soil plug is then pulverized by the sharpened bit which weighs a half-ton or more as shown in Fig. 9. The crushed material is cleaned out of the hole by means of an elongated bucket called a bailer. The surging action produced by alternately raising and lowering the bailer near the bottom of the hole washes the cuttings into the bailer,

[3] Parsons, J. and Loughney, R., 1959, "Application of Deep Wells to Dewatering New York Varved Silts." Paper presented at the American Society of Civil Engineers Convention, Cleveland, Ohio, May 4, 1959.

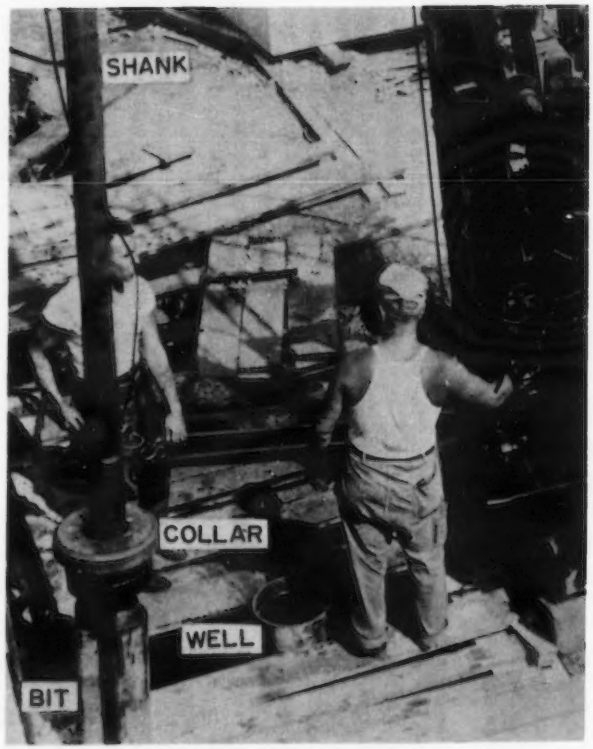


FIG. 8.—CABLE-TOOL WELL DRILLING RIG. (COURTESY OF C. W. LAUMAN & CO., INC.)

FIG. 9.—BIT USED WITH CABLE-TOOL WELL DRILLING RIG. (COURTESY OF C. W. LAUMAN & CO., INC.)

which is removed from the hole at frequent intervals and emptied. After cleaning, the casing is driven deeper and the pulverizing process repeated.

A typical rig for rotary drilling is shown in Fig. 10. A revolving bit, shown in Fig. 11, loosens and grinds the soil at the bottom of the hole. This material is flushed to the surface with a bentonite or an aqua-gel slurry pumped into the hole through a hollow drill rod. The high density of the slurry sustains the walls of the hole against collapse. After surfacing the slurry returns to a "mud ditch" or reservoir, together with its load of cuttings, and is available for recirculation. These cuttings should be collected as the well is advanced to provide a supplement to and check of the original soil data.

After the hole is drilled to the desired depth, the permanent casing is positioned in the hole, being "made up" during lowering, and the screen is set and swedged into place.

Reverse rotary drilling is a modification of the standard rotary method and speedily produces large-diameter holes usually 20 in. to 36 in. in diameter. The costly equipment and necessary mobilization operations economically limit its application to extensive dewatering projects. A reverse-rotary rig is shown in Fig. 12. The drilling procedure is similar to the standard rotary method, except that the soil is dredged up by centrifugal pump suction, and the uncased hole is sustained by a head of muddy water instead of bentonite slurry. A roller bit, shown in Fig. 13, loosens and grinds the soil at the bottom of the hole to a size that can be sucked through a hollow drill rod and pumped to a reservoir at ground surface. Stones, the size of a man's fist, can be pumped in this manner. The muddy water is recirculated from the reservoir to keep the hole constantly filled. The water level in the hole should always be maintained at least 7 ft above the natural water table to provide the necessary minimum excess head. Where the depth to the water table is less, a berm must be made to raise the rig and reservoir to the required height. After the first well is installed, it can be operated to lower the water table during the installation of successive wells, thus eliminating the berm. The permanent casing and screen are set in the same manner as described in the standard rotary manner.

Well development is an important part of well construction. This consists of removing from the formation around the screen the finer natural soil particles and any bentonite used in drilling to produce a pervious natural filter. Well development requires extensive agitation which is accomplished through the use of surge blocks, compressed air, or merely starting and stopping the pump using the falling column of water as a surge. In fine-grained formations, where the particle size is uniform, the usual methods of development do not produce wells with adequate yields. Wells in this type of soil must be provided with an artificially-placed filter around the screen. This procedure is known as "gravel-packing". Initially, a temporary outer casing is positioned by one of the three well drilling methods described earlier. The permanent inner casing and screen is then centered inside the temporary outer casing and the annular space filled with pea-sized gravel. For dewatering purposes, this packing should extend to land surface. The outer casing is then withdrawn, leaving in the hole only a short length of casing to facilitate the periodic addition of gravel as the packing consolidates. In the case of the reverse rotary method, some well drillers funnel the gravel into the open hole, which is cased only at the surface to prevent slumping and to enable the replenishment of the gravel-pack.

Pump installation follows well development. Deep-well turbines, driven by gasoline, diesel, or butane engines, or electric motors, or electric-powered

FIG. 10.—STANDARD ROTARY WELL DRILLING RIG. (COURTESY OF LAYNE-NEW YORK CO., INC.)

FIG. 11.—BIT USED WITH THE STANDARD ROTARY WELL DRILLING RIG. (COURTESY OF REED ROLLER BIT CO.)

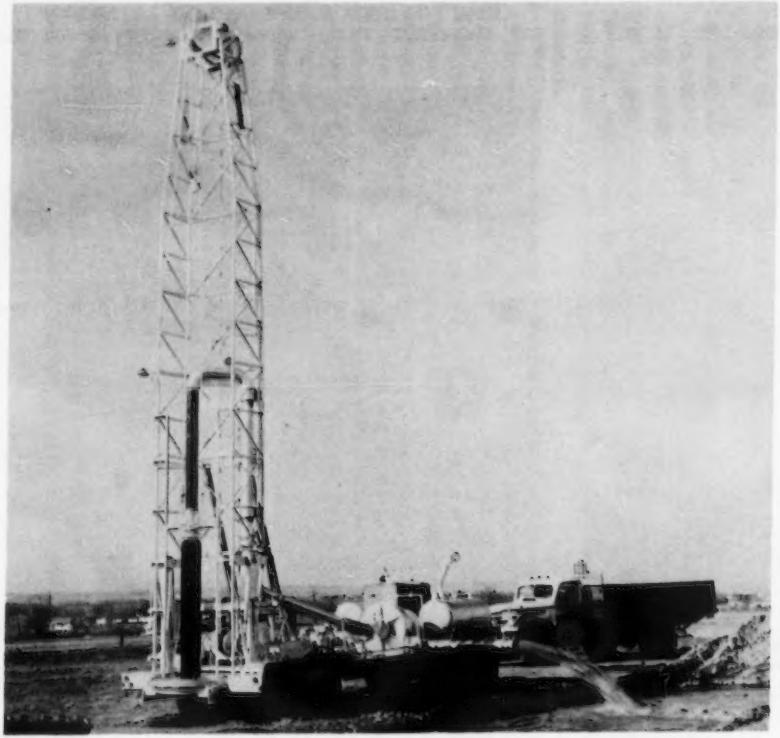


FIG. 12.—REVERSE ROTARY WELL DRILLING RIG IN OPERATION. (COURTESY OF THE WINTER WEISS CO.)

FIG. 13.—BIT USED WITH THE REVERSE ROTARY WELL DRILLING RIG. (COURTESY OF C. W. LAUMAN & CO., INC.)

submersible pumps are the types of pumps usually used. A typical deep-well turbine pump is shown in Fig. 14.

The surface housing of the turbine shown in the illustration is set directly over the well. Hung from the bottom of the surface housing is a pipe column through which the water flows to the surface. Attached to the bottom of the column is a set of pump bowls containing the impellers, which constitutes the "turbine" arrangement. Suspended below the bowls is a suction pipe with a strainer attached. Aligned with the axis of the column pipe and supported in bushings, is the drive shaft. The upper end of the shaft is connected to the prime mover, the lower end to the impellers. The prime mover at the surface, through a gear box and a packing gland, rotates the impellers at the lower end, forcing water through the pump column to the surface.

A modification of the deep-well turbine, which eliminates the bulky arrangement of the prime mover at the surface together with the appurtenant shafting, is the submersible pump. In this type of pump, the electric motor, coupled directly to the impeller through a very short shaft, is submerged below the water level. The entire arrangement is suspended from the surface by means of standard pipe, which serves simultaneously as a conduit for the pumped water. This pipe, depending on the size of the pump, ranges from 1-1/4 in. to 12 in. in diameter.

Regardless of whether a deep well turbine or a submersible pump is used, it is important to set the impellers of the pump at least 10 ft below the anticipated pumping level with all working pumps in operation. Should the water level approach the intake of the pump, erratic pump operation makes control of the surrounding ground water difficult.

INSTALLATION PROBLEMS

The foregoing discussion has depicted the installation of dewatering systems under ideal conditions. The following material describes typical methods for coping with some of the more difficult installation problems.

In the construction of sump pits—glacial erratics, cemented gravel, cobbles, and boulders, make ordinary methods of excavating and shoring difficult, and blasting is sometimes necessary. Where sheeting is too difficult, perforated caissons may be sunk to serve as encased sump pits. Constant pumping of dirty water for extended periods might gradually remove significant quantities of soil from outside of the sump area causing settlement of nearby structures. Even if not a problem of structural safety, the pumping of sandy soil can cause excessive erosion of unprotected pump parts. To prevent the pumping of soil, each sump should be constructed with adequate graded gravel filters on the walls and floor of the pit (see Fig. 1). Wood chips, leaves, and other debris may clog the suction strainer. Likewise, dredging of soil around the inlet pipe can gradually bury the strainer and critically reduce pumpage. Some provision should be made for easy raising and quick cleaning of suction pipes, hoses, and strainers.

Many wellpoint jobs are installed easily using ordinary jetting methods. However, the occasional problems that arise need special techniques and sometimes special equipment. Successful operation of wellpoints in stratified soil depends upon the creation of a continuous cylinder of filter sand around the wellpoint and riser, termed sand-wick. This wick facilitates the drainage of the water from the various strata overlying the wellpoint screen.

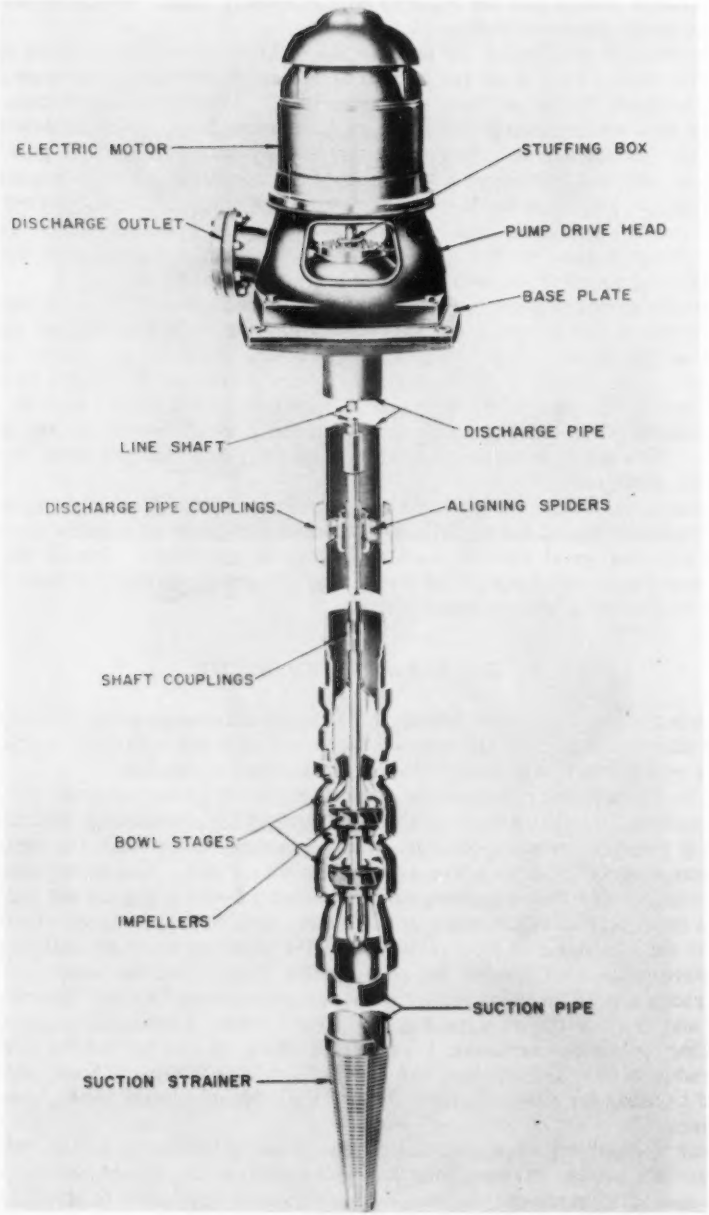


FIG. 14.—DEEP-WELL TURBINE PUMP. (COURTESY OF LAYNE-NEW YORK CO., INC.)

Selection of the proper filter material is of extreme importance. It is possible to have filter sands graded or manufactured to almost any specification. However, invariably it is more economical to use locally available stock materials. The filter material must be compatible both with the soil and the wellpoint screen. The filter sand used must effect a rapid and complete screening of the sediment from the pumped water as well as enabling the maximum yield.

Where ordinary jetting is possible and the hole will stand open, a sand-wick is most easily formed by open-sanding. After the wellpoint is jetted into place, the action of the "boil" around the riser is reduced to a gentle flow by throttling the pump. Filter sand is then poured continuously into the hole around the riser until it is filled. Where necessary, a larger and more effective wick may be formed by reaming the hole with a chain wrapped around the shank of the wellpoint, or a star-shaped reamer affixed to the lower end of the point.

The limited erosive power of water confines the self-jetting of wellpoints to soils that are comparatively loose and easily penetrated. Permeable formations with considerable resistance to penetration require the use of a holepuncher as an aid in the installation of wellpoints; this is shown in Fig. 15. The barrel of the holepuncher is about 6 in. in diameter and additional weight is affixed to the head to facilitate penetration. As the total weight of the holepuncher is about 3 tons, and varies in lengths up to 60 ft, a crane with sufficient capacity and boom length is required. After the holepuncher has been jetted into place, the cap is removed, the wellpoint assembly inserted (and sanded, if necessary), and the holepuncher withdrawn.

Only a relatively thin sand-wick is formed around the wellpoint when a holepuncher is used. Since highly stratified soils may require a larger-diameter wick to facilitate downward drainage, a combination holepuncher-sanding shell, shown in Fig. 16, has been developed. The holepuncher is as previously described; the sanding shell consists of a steel casing 8 in. to 12 in. in diameter, and a few inches shorter than the holepuncher. Soil dislodged by the holepuncher is jetted to the surface in the annular space between the holepuncher and the sanding shell. At the prescribed depth, the holepuncher is removed, a wellpoint is inserted and sanded, and the casing removed. In certain cases, the sanding shell may be used by itself. Here, a cap seals the top of the casing and a 2-1/2-in. jetting hose is attached to the casing near the top.

Even the holepuncher-sanding shell combination has its limitations when material such as thick layers of clay or dolomitic limestones, sandstones, or hardpans are encountered. In such cases, holes must be augered or drilled prior to wellpoint installation. As the usual well-drilling methods are too slow for the number of holes required, a portable drill supported on the header pipe is used. The drill rig is composed of a long hollow shaft, swiveled at the top, with a drill bit at the lower end. The shaft, which is free to slide up and down through a keyway in a turntable, is rotated by a gasoline or diesel engine. Water flows from the pump, through the hollow-shaft, and jets from the bit. Holes drilled with the rig coincide with the wellpoint design spacing. The wellpoints are then plain-jetted or installed with a casing as previously described.

Occasionally difficulty arises when jetting in soils containing highly permeable zones of very coarse sand and gravel. Characteristic of this condition is a sudden halt in penetration of the wellpoint, accompanied by a cessation of turbulence in the hole around the wellpoint. Termed "loss of boil", this is caused by jetting water escaping through the voids in the coarse strata. When the "loss of boil" is first noted, the wellpoint should be immediately raised a

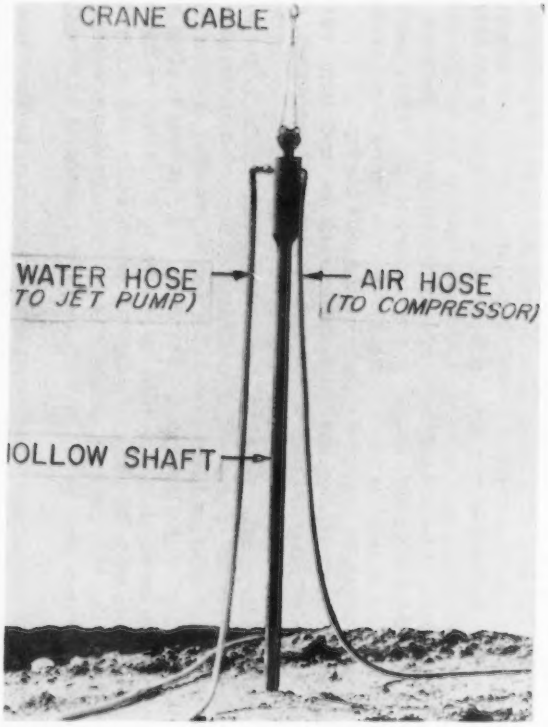


FIG. 15.—HOLEPUNCHER FOR INSTALLING WELLPOINTS.

FIG. 16.—HOLEPUNCHER-SANDING SHELL FOR INSTALLING WELLPOINTS.

few feet to prevent seizure until the turbulence reappears. The jetting operation is repeated, rotating the wellpoint and lowering it slowly as it reaches the coarse stratum. Sometimes penetration can be accelerated with a blast of air through the wellpoint.

Well-drilling rigs are designed to cope with the type of soils that ordinarily present problems in the installation of sumps and wellpoints. However, occasionally large boulders require "shooting" with dynamite to achieve penetration. Erosion of pump parts by the intrusion of silt and fine sand into a well indicates improper well installation and reflects inadequate examination of the cuttings during well drilling. Gravel-packing through pilot holes drilled to screen depth followed by extended well development can sometimes convert these wells into a usable condition. Overdevelopment or a poorly-selected screen can create a settlement problem. Instead of removing the fines only and leaving a structurally stable skeleton, the larger grains are also removed creating a subsurface void.

OPERATION OF DEWATERING SYSTEMS

Preliminary to operation, each system must be checked for air leaks. Sump and deep well systems are usually sufficiently tight when installed that only a cursory inspection is necessary. Wellpoint systems, on the other hand, have many more joints which are susceptible to air leaks. The simplest method for detecting air leaks is to close the stopcocks of the swing joints and fill the installed header pipe of the entire system with water under low pressure from a hydrant or small pump. Dripping or squirting at leaky joints is then easily located and corrected.

Actual operation of any dewatering system is relatively simple requiring only refueling, lubrication and routine maintenance. Engine oil and filters should be changed weekly or as prescribed by the engine manufacturer. Idle pumps should be operated for a few minutes every day to keep them in peak standby condition. The corrosivity of the pumped water should be checked periodically to offset serious pump and piping problems. If the ground water is known to be aggressive, all piping should be treated with protective coatings. Stainless steel, monel metal, plastic or other corrosion-resistant materials should be used in the wellpoints, pumps, and appurtenances. Extended pumping may depress the water level below the intake of wellpoints causing air to bleed into the system. Detected by periodic observation of vacuum gages placed at strategic locations in the system, such leakage may be offset by partially closing the valves in the swing joints. Screens of wells and wellpoints encrusted by exposure to waters with high concentrations of iron and calcium salts require periodic treatment with selected acid solutions. Supplemental agitation with compressed air, is often helpful in the case of wells. Below-freezing air temperatures also pose problems. Ice formed on the inner walls of the piping decreases flow if not shutting it off completely. Water-to-air heat losses can be minimized by using peripheral steam lines or by insulating the lines with straw or decaying organic material. Pump engines should be winterized with anti-freeze, and the entire arrangement housed where possible. If pumps are shut down for standby or repair purposes, they should be completely drained.

CONCLUSIONS

In conclusion, the following ten hints will help to avoid the common pitfalls in dewatering work:

1. Secure the best subsoil and ground water data available.
2. Verify these data by field test and observation before starting installation.
3. Plan equipment location carefully so that it will not interfere with construction.
4. Check sewer capacity before planning to use as a disposal point.
5. Double-check grades so that screens and sumps are set deeply enough.
6. Install piezometers for drawdown observation and check water levels periodically.
7. Test for corrosive water in a suspicious environment on a long-term pumping job.
8. Anticipate freezing weather. Summer installation might imply winter operation.
9. Keep standby pumps connected and ready for immediate operation.
10. Instruct personnel thoroughly in the operation and maintenance of the dewatering system. Pumping is usually around-the-clock and 7 days per week.

Successful dewatering is no coincidence. It is the teamwork of sound engineering, farsighted planning, painstaking installation, skillful operation, and diligent maintenance.

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

Page

Design and Construction of Ambuklao Rockfill Dam, by E. Montford Fucik and R. F. Edbrooke. (December, 1959. Discussion: June, 1959. Discussion closed).
by E. Montford Fucik and R. F. Edbrooke (closure) 69

A Review of the Engineering Characteristics of Peat, by MacFarlane. (February, 1959. Prior discussion: June, 1959. Discussion closed).
by Philip Keene 71

Physico-Chemical Properties of Soils: Ion Exchange, by A. W. Taylor. (April, 1959. Discussion: April, 1959. Discussion closed).
by A. W. Taylor (closure) 73

Physico-Chemical Properties of Soils: Properties of Soil-Water Systems, by I. Th. Rosenquist. (April, 1959. Discussion closed).
by I. Th. Rosenquist (closure) 75

Physico-Chemical Properties of Soils: Role of Soil Technology, by T. William Lambe. (April, 1959. Discussion: October, 1959. Discussion closed). . . . . ---*

Compressibility as the Basis for Soil Bearing Value, by B. K. Hough. (August, 1959. Prior discussion: December, 1959. Discussion closes February, 1960).
by Ralph B. Peck 79
by Clifton E. R. Lawson 80
by Douglas R. Brown and William M. Crawford 81
by H. O. Ireland 82
by Benjamin Koo 84
by Alfred C. Sheer 86

Note.—This paper is a part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, SM 1, February, 1960.

*There will be no closure.

Investigation of Underseepage-Mississippi River Levees, by W. J. Turnbull and C. I. Mansur. (August, 1959. Prior discussion: none. Discussion closed).
by Max Suter 87

Rock Characteristics of the Paulo Alfonso Power Plant, by Pichler and de Campos. (August, 1959. Prior discussion: none. Discussion closed).
by A. A. Eremin. 89

Construction Materials Control-Aascho Road Test, by James F. Shook. (October, 1959. Prior discussion: none. Discussion closes March, 1960).
by Edward A. Abdun-Nur 91

Rate of Constructing Embankments on Soft Foundation Soils, by Herbert L. Lobdell. (October, 1959. Prior discussion: none. Discussion closes March, 1960).
by W. H. Gotolski 93

Major Power Station Foundation in Broken Limestones, by Swiger and Estes. (October, 1959. Prior discussion: none. Discussion closes March, 1960).
by Ralph B. Peck 95

Design of Underseepage Control Measures for Dams and Levees, by W. J. Turnbull and C. I. Mansur. (October, 1959. Prior discussion: none. Discussion closes March, 1960).
by Marcel Bitoun and Jorgen Christiansen 99

DESIGN AND CONSTRUCTION OF AMBUKLAO ROCKFILL DAM[a]

Closure by E. Montford Fucik and R. F. Edbrooke

E. MONTFORD FUCIK[1] and R. F. EDBROOKE,[2] M. ASCE.—The head developed at Ambuklao consists of (a) the head developed by the dam and, (b) the head gained by having the tail tunnel discharge to the river at a point a considerable distance from the dam, thus taking advantage of the river gradient. Either an above-ground powerhouse or an underground powerhouse located at the point of discharge of the tail tunnel, thus achieving the same head as in the adopted design, would have required penstocks or pressure tunnels to carry the water to the powerhouse. Cost estimates indicated that the location of the powerhouse under the dam, which required the minimum length of pressure tunnel and penstock, was the most economical solution.

The depth of the river gravels varied from 15 ft to a maximum of about 45 ft.

No attempt was made to measure the pore pressure in the test fills.

Seepage along the abutment contact is one of the critical areas of design and must be of prime consideration in a dam of this magnitude.

In the design of Ambuklao Dam one of the prime considerations was to obtain a design which would provide adequate protection against seepage along the abutment contact area. At the conception of the Ambuklao project there was little precedence for a vertical core dam of this magnitude and no precedence as a sloping core dam. It was concluded that, from both the engineering and economic standpoints, the optimum design would be provided by the vertical core.

A vertical core permits a larger area of contact between the foundation and the core than does a sloping core, and the contact area is also under greater vertical pressures in a vertical core dam. This feature was a major consideration leading to the adoption of the vertical core.

The settlement curves shown in Fig. 7 comparing the diorite and metamorphic rock crushing were not used for prediction of settlement. The test was for qualitative rather than quantitative purposes. Further, the variation in gradation between the rock tested and rock used in the fill made questionable the accuracy of any settlement computations made from this data. The writers, however, recognize the potential use of such data after more experience has been gained comparing test data with prototype action.

The writers would like to point out a misleading impression that may have resulted from Fig. 10 of their paper. The section of the dam illustrated in

---

[a] December, 1959, by E. Montford Fucik and R. F. Edbrooke.

[1] Exec. Vice Pres., Harza Engrg. Co., Chicago, Ill.

[2] Asst. Proj. Mgr., Harza, Engrg. Co., Chicago, Ill.

Fig. 10 appears to be the maximum dam section; however, the sample computations represent an analysis of a section on the left abutment of lesser height. The purpose of the figure was to illustrate the method of analysis rather than a specific mathematical analysis of the maximum section.

A REVIEW OF THE ENGINEERING CHARACTERISTICS OF PEAT[a]

Discussion by Philip Keene

PHILIP KEENE,[1] M. ASCE.—The author is to be congratulated on his thorough work of assembling the material from the literature available to him on the subject of peat and presenting it in various categories used frequently in the field of soil mechanics. The paper is particularly valuable because of the large amount of numerical values given. This subject, however, is difficult to present in coherent form because of the wide varieties of peat material and wide variations in degree of decomposition of the vegetal matter, in amount of foreign mineral matter, in percentage of water content and in degree of drying, especially at the surface. As a guide to the varieties of peat material to be found in Canada the author's own work (Ref. 25) is an excellent reference. It gives a detailed classification of peats, and also a table of muskeg coverage classes and a table of topographic features with the peat class usually found at each type of topography.

The writer has had some experience with the engineering properties of peats found in Connecticut, and will present herewith some data to increase the engineering information on peats which the author rightfully states is rather scarce at present.

Connecticut's peats are of two general types. One type is found in upland swamps and consists chiefly of decomposed leaves, twigs and roots of trees and bushes, and grass and grass roots; they are usually brown in natural color but turn black when exposed to the air. Sometimes these peats are finely decomposed so that identification of the particles requires careful scrutiny with the naked eye. The other type is found in existing or former tidal marshes and consists chiefly of marsh grass roots and reeds, usually with considerable silt and clay materials; this type often changes with increased depth to organic silt-clay.

Ash content of the upland peats varies from about 8% to 40% ash for the samples which were classed as "peat"; samples classed as a mixture, such as "peat and silt," have ash contents varying from 50% to 65% ash. The tidal peats vary widely in ash content, depending on the mineral content, from about 8% ash for a "cotton grass peat" to 75% for a "silt-clay with peat." Ash contents were determined by burning in an electric furnace (muffle) at 3000° F.

Absolute specific gravity of the solid particles in the upland peats varies from about 1.2 to 2.0 and the corresponding density of the saturated peat mass is from 63-1/2 to 68 lb per cu ft. These values are for the peats with ash content ranging from about 10% to 50%, respectively. The specific gravities and the densities are both determined by weighing saturated chunks ejected

---

[a] February, 1959, by Ivan C. MacFarlane.

[1] Engr. of Soils and Foundations, Conn. State Highway Dept., Hartford, Conn.

from large-diameter thin-wall undisturbed sampler tubes. Immediately after ejection, a chunk is placed in a hair-net and quickly weighed in the air and then weighed submerged. It should be noted that among the mass densities quoted by the author are some less than that of water. If those values are correct, those peats are either above the water table or, if below the water table, tend to float and are held down by the overburden or by a root structure anchored below.

In the matter of settlements, the writer finds that settlement estimates based on conventional consolidation tests on good, large-diameter undisturbed samples give quite good agreement with field observations. Preconsolidation load obtained by using the Casagrande method on the void ratio-log pressure curve gives close agreement with the calculated overburden load for normally-consolidated peat. Settlement calculations from the virgin compression portion of the e-log P curves gave good agreement in the cases where reliable settlement information was obtained. In this connection it should be mentioned that the values for the compression index, C_c, which is the slope of the virgin compression line on the e-log P curve, is in quite good agreement with the approximate value of $C_c = 0.01\ W\ \%$, where W denotes the natural water content in percent of dry weight, shown in Plate No. 46 in the excellent publication on vertical sand drains.[2] Possibly this relationship is similar to the one quoted by the author from Cook (Ref. 3), although the latter uses the expression "coefficient of compressibility" rather than "compression index." In practice the writer finds that in estimating settlement in peat, using the conventional formula

$$H = H_o \frac{C_c}{1+e_o} \log \frac{P_f}{P_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which H is the consolidation of an element of peat, H_o represents the original thickness of the element of peat, C_c is the compression index, e_o denotes the original void ratio, P_f is the final vertical compressive stress, and P_o represents the original vertical compressive stress. The value of $C_c/1+e_o$, is between 0.6 and 0.45 for peat with low ash contents such as 10% to 30%. This is in contract to tidal organic silt-clays where the value of $C_c/1+e_o$ is 0.3 to 0.25.

In the matter of rates of settlement, data are less reliable, as obviously the analyses are more difficult than for total amounts of settlements. The question of the long-time secondary settlement is of considerable importance in maintenance problems and in questions regarding the time to remove an overload during construction. The writer's data from some consolidation tests on upland and tidal peats and on long-time settlement observations on tidal peat projects indicate that after primary consolidation is about completed, secondary consolidation proceeds as a straight line on the settlement-log time graph. This phenomenon was noted in References (1), (20) and others. The slope of the secondary line gives a settlement, in one cycle of time, equal to about 10% to 20% of the primary settlement. For example, if primary consolidation was 4 ft and was completed in 1 yr after application of the load, secondary consolidation would cause a settlement of about 0.4 to 0.8 ft between 1 yr and 10 yr, and an additional 0.2 to 0.8 ft between 10 yr and 100 yr. However, it is often difficult to determine from the data at what point the primary settlement is about completed and the secondary settlement has begun.

[2] "Study of Deep Soil Stabilization by Vertical Sand Drains," by Moran, Proctor, Mueser and Rutledge, for Bureau of Yards & Docks, U. S. Dept. of the Navy, June 1958.

PHYSICO-CHEMICAL PROPERTIES OF SOILS: ION EXCHANGE[a]

Closure by A. W. Taylor

A. W. TAYLOR[1].—In regard to Dr. Low's comments, it should be emphasized that the principal reason why the electrical double layer theory fails to give a complete picture of the physical behavior of soil clays is that it is based upon too simple a physical model for this purpose. As Alan S. Michaels has pointed out,[2] there is ample evidence that even mineralogically pure clays cannot be regarded as simple negatively charged colloids, and mutual interaction of positive and negatively charged sites on the colloid particles being the principal factor controlling the strength of clays in the flocculated state. It would seem hardly likely that any theoretical treatment of the swelling process in clay systems will be at all successful unless this mechanism is taken into account.

While the complexity of the real physical picture thus limits the detailed application of the theory, it does give a very satisfactory interpretation of the effects of electrolyte concentration and ionic valency on the strength and plastic behavior of clay systems. This treatment is well illustrated by the classic work on the flow properties of kaolinite clays by Schofield and Samson, which has been referred to previously by both Dr. Michaels and the writer. This study is more remarkable in that it is one of the few published in which adequate precautions were taken to remove the exchangeable aluminum from the natural clay so that the dispersion was not hindered by the flocculating power of this ion or by the binding effect of iron or aluminum hydroxides precipitated within the clay mass.

The discrepancy which exists between the mathematical model on which the electrical double layer theory is based and the complexity of real systems is not, however, confined to the departure of soil clays from the ideal state of a simple negatively charged colloid. In the derivation of the theory, as discussed by Mr. Low, the ions are assumed to be without finite size and to show no polarization or mutual interactions. Until suitable corrections can be applied for these perturbations it is unrealistic to expect the theory to give a precise description of the differences in the distribution of two separate ionic species within the diffuse layer. Such failures do not, however, bring the validity of the basic approach into question, since they arise from our ignorance of the true character of the ions themselves. Very similar difficulties are met in the Debye-Huckel theory of the nature of electrolyte solutions, which has close analogies with the electrical double layer theory, and further

---

[a] April, 1959, by A. W. Taylor.

[1] Dept. of Ceramic Tech., Pennsylvania State Univ., University Park, Pa.

[2] Alan S. Michaels, Discussion of "Physico-Chemical Properties of Soils: Properties of Soil-Water Systems," by I. Th. Rosenquist, Proc. paper 2010, April, 1959.

progress in the refinement of the latter cannot be expected until the physical chemistry of solutions of strong electrolytes has been worked out in more detail. Even as it stands at the present time the theory does however provide a satisfactory interpretation of the effects of ionic valency and salt concentration on the stability of colloid systems.

The various other treatments of ion exchange discussed by Mr. Amorocho were omitted from the original paper because the double layer theory is the only one which predicts that any relationship is to be expected between the exchangeable ions of the clay and its mechanical stability. Most of these treatments were developed in connection with studies of the retention of plant nutrients in the soil, and bear no relation to the mechanical stability of the system except in the special case where problems arise in the reclamation of saline soils under irrigation. Of the various schemes discussed by Mr. Amorocho, the Gapon equation - which can be regarded as a special case of the double layer equations where the exchangeable ions are confined to a very shallow layer close to the charged surface - has been found to give a reasonably adequate picture of the connection between the composition of the ambient solution and the quantities of ions held on the exchange complex for clays of a high surface density of charge. This equation cannot however be used to explain changes in the physical state of the soil without recourse to some type of double layer interpretation.

In summary it may be said that the effects which the exchangeable ions of a soil exert upon its physical properties are due to the influence which these, together with the salt concentration, have upon the forces of mutual repulsion between the clay particles. As predicted by the theory of the electrical double layer the ions which are most tenaciously held in the exchange complex are those which possess the strongest flocculating power and tend to promote soil stability.

However, this theory cannot be extended to give a complete and detailed description of the mechanical properties of a soil since these depend upon factors which the theory does not take into account, such as the attractive forces existing within arrangements of structures of bipolar particles which are responsible for the mechanical properties of a stable flocculated soil. A better understanding of these properties is more likely to be obtained through careful studies of the distribution of electrical charges carried by soil clays along the lines pioneered by R. K. Schofield and the late H. R. Samson rather than by any treatment based upon the idea that a soil clay can be regarded as a simple negatively charged colloid.

PHYSICO-CHEMICAL PROPERTIES OF SOILS: SOIL WATER-SYSTEMS[a]

Closure by I. Th. Rosenquist

I. TH. ROSENQUIST[1].—Dr. Michael's discussion has been most valuable and the writer is thankful for his comments. Some points may, however, be commented on:

Exchangeable Cations, Swelling, and Liquid Limits of Clays.—Michael states that the effect of cations leads to a decrease of the interlayer sorption of water in the order Li > Na > K > Rb > Cs in minerals with expanding lattices. Further that the liquid limit of these clays decreases in the same order. There is a slight difference in opinion here. Probably more correct is to say Li > Na > K ≈ Rb ≈ Cs. The liquid limits however go Li > Na > K < Rb < Cs. The effect of the same ions on the non-expanding minerals, say illites, is that the liquid limits go Li < Na < K < Rb < Cs. This does not mean, however, that an exchange of a small cation by a larger and more polarizable cation in an undisturbed state will lead to an increase in the shear strength of an illitic clay and a decrease in a montmorillonitic clay. On the contrary, experiments carried out during the last three years at the Norwegian Geotechnical Institute prove beyond doubt that if a sodium montmorillonite sediment is consolidated and kept under a constant consolidation load and then perculated by a potassium chloride solution, the undisturbed shear strength of the sediment increases considerably, whereas the remoulded strength decreases. Thus the sensitivity increases from a relatively low value to sensitivity values above 10. The liquidity index increases considerably as the liquid limit drops and the water content is kept constant. This finding indicates to the author that the shear strength of a sediment in its undisturbed state is a function of the adsorbed ions in the areas of "quasi-contact," that is where the corners or edges of one mineral are close or in contact with the planes of another mineral. By a replacement of sodium by potassium this bond strength increases although the internal swelling of the montmorillionite mineral decreases. A relatively thick sodium montmorillonitic plate is transformed into a thinner potassium montmorillonitic plate and the amount of external water is increased by the amount lost between the layers. By non-expanding lattices which were the cases dealt with in my lecture, no such expulsion of internal water takes place and the replacement by a larger cation leads to an increase in the remoulded strength as well as in the undisturbed strength, and an increase in the liquid limit and a reduction in the liquidity index.

---

[a] April, 1959, by I. Th. Rosenqvist.

[1] Prof. and Hd., Geochemical Div., Norwegian Geotechnical Inst., Univ. of Oslo, Oslo, Norway.

Effect of Electrolytes on the Properties of Clay Suspensions.—The writer fully agrees with Dr. Michaels in his view points, but would like to point to the fact that not only the cations of the electrolyte content in the water phase is of importance, but also the anions. The zeta-potential and all its derivaties depend upon the cations and anions, even in case of anions which are normally not considered as dispergeants, for instance the zeta-potential decreases slower by increasing concentration of sodium sulphate than by sodium chloride, the same is the case with other cations. For instance when phosphate, silicates or carbonates are added, the influence is still more pronounced. The surface charge density is primarily a function of the pH as stated by Dr. Michaels, but the variations in pH will normally in clays dealt with by the author vary within fairly narrow limits, and may as an approximation be regarded as constant, whereas the concentration of other ions than hydronium may vary considerably.

Cementation Processes in Consolidated Clays.—Even an artificial clay sediment where no cementation seems to have taken place has most of the properties of natural clays. The importance of such cementation in young illitic sediments is, of course, not to be entirely neglected, but the effect of other variables seems to be more interesting. That is the reason why this cementation was considered less important.

Effect of Cations on the Sediment Volume of the Clay.—It is agreed upon that aniline is a far weaker base than cyclohexylamine. By the concentrations used the buffer capacity of the clay suspension will, however, reduce this effect upon the pH value of the suspension, and the final pH is not so different as should be expected from the dissociation constants as Dr. Michaels seems to think.

The State of Adsorbed Water of Clays.—The state of adsorbed water of clays is still not fully understood, neither by the author nor, as far as literature goes, by any other. Concerning the influence of the tortuosity factor it is difficult to understand how a factor of the order 20 could be correct in flocculated clay sediments of fairly high water content. In the diffusion coefficient referred to by the author a tortuosity factor of $\sqrt{2}$ has already been introduced. This may perhaps be a little low, but is in accordance with a completely haphazard arrangement of indefinitely thin plates. Even though the absolute values of the diffusion coefficients are of great interest, the relative values and the temperature effect is of greater importance and the only factors necessary for a calculation of activation energy etc.

As to the conclusions drawn from the work by Dr. Michaels and Lin (1955), the author is aware of the fact that the same conclusions have not been drawn by the original authors. To give a different interpretation of observed facts is, however, the normal procedure in the development of science.

As to the role of water in clay cohesion there seems to be no major disagreement between Dr. Michaels' view and the present view of the author. Neither the author nor Dr. Michaels believe in the adsorbed water as "the glue." The consequence of very many different experiments indicate that the physical properties of the innermost adsorbed water differ from normal water and both the entropy value and the diffusibility approach more or less the

values of a crystalline material, viz. ice. Whether this water really represents a solid or highly viscous material is still open for discussion. The influence of this part of the water-phase is certainly an important factor contributing to the strength of a soil. The secondary settlements and creep properties are much more pronounced in colloidal soils than in sand and gravel. This difference may be explained according to a diffusion model in the "solid" water at the points of contact between minerals.

COMPRESSIBILITY AS THE BASIS FOR SOIL BEARING VALUE[a]

Discussions by Ralph B. Peck, Clifton E. R. Lawson,
Douglas R. Brown and William M. Crawford, H. O. Ireland,
Benjamin Koo, and Alfred C. Sheer

RALPH B. PECK,[1] F. ASCE.—This paper constitutes a forceful reminder that the limitation of differential settlements among the footings of a structure should be a prime concern of the designer and should not be considered less important than provision of an adequate factor of safety against rupture. To this thesis no objection can be raised. The writer is of the opinion, however, that the case has been overstated. He questions whether there is as strong a tendency as the author implies toward placing major emphasis on elimination of the possibility of rupture and neglecting the importance of settlement. Nor has he noted the "growing tendency to consider that allowable soil bearing value is related principally to shearing strength. . . ." On the contrary, he believes most teachers of soil mechanics and a great many designers have for a number of years been giving at least equal weight to the requirements of settlement and rupture. In the case of footings on sand, indeed, it has been emphasized[2] that the settlement requirements commonly govern the design and that, except in unusual circumstances, safety against rupture can be taken for granted.

However, even if one accepts the author's thesis, the analyses, which purport to be entirely general as to soil type, lead to erroneous conclusions with respect to certain problems of practical importance, especially in connection with footings on sand.

The author concludes, specifically, that, "for a given settlement, the contact pressure for a footing of given size and depth in an open excavation without surcharge may be greater than for a surcharged footing." This statement is at best incomplete and may be misleading: its implications require examination.

If we consider two identical foundations, established at the same level, at the same soil pressure, on the surface of the same sand, we can expect the settlements also to be equal. If then a surcharge is placed around the footings of one of the foundations, that foundation will experience additional settlement on account of the additional load. For this rather artificial circumstance, which presupposes the column loads to be acting before the surcharge is placed, the statement is true (although even in this case the additional settlement due to the surcharge is likely to be quite uniform).

---

[a] August, 1959, by B. K. Hough.

[1] Prof. of Foundation Engrg., Univ. of Ill., Urbana, Ill.

[2] See, for example, "Foundation Engineering," by Peck, Hanson and Thornburn, p. 226; "Soil Mechanics in Engineering Practice," by Terzaghi and Peck, p. 438.

On the other hand, if the surcharge around the footings is placed before the columns are loaded, or if the footings are constructed in pits in the surrounding surcharge, then the settlement of the surcharged footings, due to the application of the building loads, will be less than that of the footings without surcharge. Since this the usual case of practical importance, it suggests that the author's general conclusion is invalid or, at least, is too sweeping. It also suggests that the designer must discriminate between settlements that are significant and those of no consequence. Settlements of footings, prior to the time the structure begins to act as a unit, are usually of no concern.

Some of the incorrect conclusions concerning footings on sand arise from the author's treatment of all types of soil as having similar stress-deformation characteristics. For sands, particularly, certain of the assumptions are rather unsuitable. The author considers that the straight-line portion Oa of the load-settlement curve, Fig. 1(a), is due chiefly to one-dimensional soil compression, and that the shearing stresses associated with this portion are relatively small and lateral strains negligible. For a loaded area on sand without surcharge this is by no means true. Even in an elastic solid, unless Poisson's ratio is zero, an appreciable part of the vertical settlement may be attributed to lateral strains; indeed, if Poisson's ratio equals 0.5, all the settlement is associated with shearing deformations. Thus, significant shearing deformations are not reserved for states approaching rupture. Nor does a straight-line load-settlement relation imply lack of shearing distortions.

Moreover, if a sand is surcharged, its compressibility decreases markedly as a consequence of the increase in all-around pressure. This increase in rigidity due to confining pressure is one of its most significant properties of a sand. It has a decisive influence on the settlement, but is is ignored in the author's development. For this reason, the author's analytical procedures can hardly be considered acceptable, at least for cohesionless soils.

Finally, the author points out that shear tests for calculation of the rupture load are too expensive for many routine jobs. The same objection may be raised to tests for compressibility. The author, however, uses the standard penetration test as an index to the compressibility; it can be used equally well as an index to the strength. Both usages depend on statistical correlations and, when used with judgment, are appropriate on routine or small projects.

CLIFTON E. R. LAWSON,[1] A.M. ASCE.—The author is to be complimented for his rational approach to the design of spread footings. It is interesting to see how building codes, which are based on years of experience, tend to substantiate his opinion that consideration of settlement is more pertinent to the problem than is consideration of shear failure.

There appears to be too much emphasis by some engineers on the determination of the "strength" of the soil rather than the compressibility of soil. Those who concentrate their efforts on shear failure design methods are bypassing the real issue. However, up to now they have not had a workable method available for directly designing a footing to meet a given tolerable settlement. Now, by using Mr. Hough's method, a designer can go directly to the heart of the footing design problem - what size footing for "X" inches

---

a August, 1959, by B. K. Hough.

1 Soils Engr., Central office, Wis. State Highway Comm.

tolerable settlement? This new method does away with extraneous computations. The approach of designing by a safety factor against shear failure and then "checking" for settlement is placing the cart before the horse.

The designing structural engineer sometimes has been slow to accept the complicated footing design charts sometimes furnished them by soils engineers. The preference has always been to have the soils engineer furnish a single value of soil bearing capacity for any size footing. Now with a very minimum of extra effort any designer should be able to produce a well engineered spread footing design.

It is hoped the author will continue his work and present charts and tables to include the following situations; footings on several soil layers, wall footings with unbalanced surcharge, footings subjected to moments and footings on compacted fills of narrow width. The latter case is of particular interest in connection with grade separation structures for expressways.

The writer is interested in knowing if Mr. Hough has had any of the lengthy computations set up on an electronic computer. If this is possible, many different footing situations could be investigated and it is conceivable that curves similar to those already presented could be worked out. Practicing soils engineers will readily recognize the advantages of the procedures already proposed by Mr. Hough. Any added work in this general direction is bound to be just as popular.

DOUGLAS R. BROWN,[1] A.M. ASCE, and WILLIAM M. CRAWFORD[2].—This fundamental and yet original paper is definitely a valuable contribution to our understanding of the action of spread footings. The basic premise is that differential settlement of spread footings, rather than shear failure, is actually the controlling factor for most structure foundations. A corollary is that the process of making settlement estimates is frequently neglected with relation to ultimate bearing capacity calculations. In these two thoughts, the writers agree wholeheartedly.

Mr. Hough's equations appear straight-forward, but limitations of one of the basic assumptions on which his derivations are based could use some elaboration. It is stated "When compressibility data are presented in semi-logarithmic form as shown in Fig. 2(a), there are a number of situations in which the compression diagram is approximately linear over at least a limited range of loading." Although the resulting equation leads to the generally accepted equation,

$$\Delta H = H \frac{C_c}{1+e} \log \left(1 + \frac{\Delta p}{p_i}\right)$$

a brief discussion of the actual usual limits of linearity would be pertinent to the reliability of Mr. Hough's further development.

---

[a] August, 1959, by B. K. Hough.

[1] Senior Engrg. Geologist, Los Angeles County, Calif, (Formerly Senior Engrg. Geologist, Bridge Dept., Calif. Div. Highways).

[2] Jr. Civ. Engr. Bridge Dept., Calif. Div. of Highways, Los Angeles, Calif.

It is believed that where a comparison is invited between Eq. 27 and 24, that the author intends Eq. 20 rather than Eq. 24. (A comparison with Eq. 24 is made where the depth is again 1/3 of the depth of significant stress.)

One of the most interesting items in Mr. Hough's paper appears somewhat unrelated to the rest of the paper. This is Fig. 3 which gives the approximate bearing capacity index, C_1, for different values of standard penetration resistance, with reference to different soil compositions. Since the standard penetration sampler measures primarily shear strength, this chart correlates shear strength and consolidation or settlement characteristics. In spite of statements of many authorities that there is no significant relationship between shear strength and compressibility, the Bridge Department of the California Division of Highways has for some time operated on the theory that, with soil composition and grain size distribution approximately constant, there is a correlation between shear strength and compressibility. Mr. Hough's graph not only indicates the same principle, but in at least a rough way, gives actual values of correlation. Correlations of the Bridge Department have had to be derived from field results where soils are seldom uniform. This leads to trial-and-error procedures with resulting inaccuracies. It is assumed that Mr. Hough derived the values, indicated by the curves in the graph, in a more scientific manner, and a brief description of his methods would be welcome.

Although the paper was intended to apply to settlement under footings, the principles are general. Therefore, the values of the graph (Fig. 3) together with Eqs. 3 and 6 and a Bousinessq stress distribution were applied to settlement under highway fills. Cases chosen were adjacent to bridges in Southern California where standard penetration tests had been made and where settlement measurements (ranging from 2 in. to 1 in.) had been made. Soil classification was based on inspection rather than testing. The calculated results were then compared to the actual measured settlements of original ground under the fill load. Just recently in order to expedite the calculations, the problem has been programmed for an IBM 650 computer. Data sheets for the computer can be completed in less than five minutes.

More than forty comparisons have been made to date and many more should be made in the next few months. For those cases where the soil was dominantly sand and the water table was low, the Hough curves gave excellent results. Out of about twenty comparisons, the maximum error was only 10% in approximately 70% of the cases. The maximum limits of error were about 40%. Where there was sand with a high water table (ten cases), the estimate was usually about 30% to 40% lower than the measured value. In the case of clayey soils, where more than fifteen comparisons were made, results ranged from exact agreement to (-)50% error. The estimates averaged about 40% low in ten cases for this soil condition.

Conclusions resulting from this program are somewhat premature, but the foregoing results are given to indicate that the Hough curves show considerable promise, particularly where a fast, and approximate, estimate is desired. An interesting fact is that for a given general type of soil, the discrepancies tended to be grouped, so that better agreement might be obtained by some adjustment of the curves.

H. O. IRELAND,[1] M. ASCE.—A rather interesting paper has been prepared which presents a method for computing an allowable soil pressure corres-

[a] August, 1959, by B. K. Hough.

[1] Prof. of Civ. Engrg., Univ. of Illinois, Urbana, Ill.

ponding to a predetermined magnitude of permissible settlement for footing foundations on various materials. As stated by Mr. Hough it is true that many present-day analytical procedures for the determination of soil bearing value do require evaluation of the shearing strength and ultimate bearing value of the supporting material as an initial step. This is merely to say that we must know the ultimate strength of the material with which we are dealing.

The author has indicated that an analysis of settlement due to soil compression, if undertaken at all, is often regarded as a secondary rather than primary operation and one which is of interest chiefly with clays. The writer believes that the settlement analysis is not a secondary operation but is an independent operation not related to the allowable bearing value as commonly used. Once a maximum allowable bearing pressure has been determined based upon strength, an independent computation of the settlement may be made. If it turns out that the settlement that would be produced by the maximum allowable bearing value based on strength is larger than can be accepted, a reasonable procedure would be to reduce the soil pressure to some lower bearing value and again compute the corresponding settlement.

Insofar as soils other than clays are concerned, it may be noted that the design procedures for foundations on sand as recommended in "Foundation Engineering" by Peck, Hanson and Thornburn are based upon considerations of settlement almost entirely with only a routine check of bearing capacity in order to insure that this is adequate.

The author makes use of a term in his analysis which he calls the bearing capacity index $C = 1 + e/C_c$. It is not clear to the writer what value of C_c is intended to be used in this relationship. The writer would assume that this value is the one corresponding to the corrected consolidation test simulating the field condition which is generally accepted in the literature as the compression index, C_c. Thus there is only a single value of C_c for a given material. If this is intended, then the value of C_c should be determined from the field. The author implies that C_c is more easily obtained than the shear strength of the soil. The writer is inclined to believe that an accurate determination of C_c is considerably more difficult than the determination of the shear strength for most materials. It involves the best of sampling and careful laboratory consolidation tests.

Any procedure which utilizes C_c is of necessity limited to soils where C_c is a useable measure of the performance of the material. The author has made no distinction between normally loaded clays of high sensitivity, of low sensitivity, of heavily preloaded clays or of sands in the application of his proposed determination of soil bearing values. Since C_c is not applicable to undisturbed samples of soils of high sensitivity, inasmuch as their field behavior is not characterized by a straight line relationship, this method should not be used for highly sensitive clays. In the case of preloaded, or heavily over-consolidated, clays the author's procedure would seriously underestimate the allowable bearing value unless the value of C_c is modified somehow to suit the particular circumstances. This is because the slope C_c of the e-log curve for a preloaded clay begins at the maximum previous overburden pressure whereas the applied load Δp causes a stress increase above the present overburden pressure p_o. In some cases $p_o + \Delta p$ is less than the maximum preload. With respect to sands, very little information is available concerning C_c. However, the problem of obtaining a truly undisturbed sample representative of the material in the field is not commonly successful.

In conclusion, it is the writer's viewpoint that we are seldom justified in attempting to design individual footings for precise pre-determined settlements. The subsurface exploration program should indicate the relation range of variation of soil properties to be expected but can hardly be expected to provide enough information to permit precise settlement computations for every footing to be considered. No matter how complete the program of soil exploration and testing may be there always remains a large margin of uncertainty concerning the exact nature of subsurface conditions at a given site. In view of this, it seems illogical to attempt to proportion individual footings based on precise settlement computations.

BENJAMIN KOO,[1] M. ASCE.—Mr. Hough has taken a valuable step towards footing designs by emphasizing equal settlement due to soil compression rather than to rupture analysis. It is hoped that this paper will lead to a recommended practice for general use.

With reference to Fig. 4, it is very common that structural loading includes a uniformly distributed load in addition to a column load as illustrated in Fig. 4-I. For the sake of practical interest, equations applicable to this type

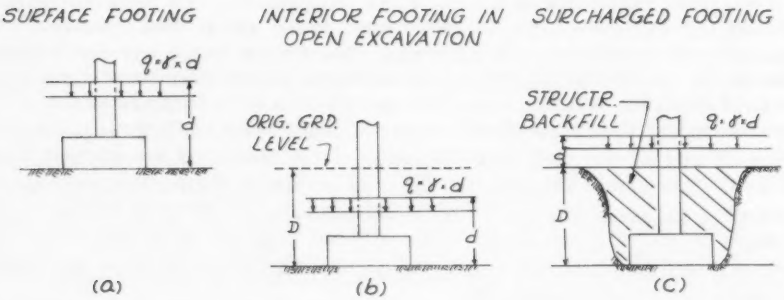


FIG. 4-I

of loading and supplementary to author's basic equations, (Eqs. 10, 11, 13, 14, 20, 22, 24, 25, 27, 28, 29, and 30), are shown.

Let $$q = \gamma d \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (a)$$

in which q is the uniformly distributed load (pounds per square foot) such as weight of floor slab, machine load on floor, fill under slab, etc., γ denotes the unit weight of soil (pounds per cubic foot), and d is the equivalent height of soil (feet).

CASE I. Surface Footing. Fig. 4-I (a).—The average value of stress increment at depth h can be written in the form

---

[a] August, 1959, by B. K. Hough.

[1] Structural and foundation engr., M. H. Treadwell Co., New York, N. Y.

$$\Delta P = \frac{B^2}{(h + B)^2} \quad p_c + \gamma\, d \quad \cdots\cdots\cdots\cdots \quad (10a)$$

The depth of significant stress is expressed as,

$$h_s(h_s + B)^2[1 - 10\,d/h_s] = \frac{10B^2}{\gamma} \quad p_c \cdots\cdots\cdots \quad (11a)$$

At one-third depth, h_s, the average stress increment is,

$$\Delta P = \frac{9B^2}{(h_s + 3B)^2} \quad p_c + \gamma\, d \cdots\cdots\cdots\cdots \quad (13a)$$

The particular value of the contact pressure p'_c within a specified settlement s can be shown as follows,

$$p'_c = \frac{\gamma}{27\,B^2} \quad H[10\,S_c/H - (1 + 3d/H)]\,(H + 3B)^2 \cdots\cdots \quad (14a)$$

CASE II. Interior Footing in Open Excavation. Fig. 4-I (b).—Following the same sequence as for the previous case;-

$$\Delta P_{NET} = \frac{B^2}{(h + B)^2} \quad p_c - \gamma\, D + \gamma\, d \cdots\cdots\cdots \quad (20a)$$

$$[h_s + 11\,D - 10\,d]\,(h_s + B)^2 = \frac{10\,B^2}{\gamma} \quad p_c \cdots\cdots\cdots \quad (22a)$$

$$\Delta P_{NET} = \frac{9B^2}{(h_s + 3B)^2} \quad p_c - \gamma\, D + \gamma\, d \cdots\cdots\cdots \quad (24a)$$

$$p'_c = \frac{\gamma}{27\,B^2}[(H + 3D) \cdot 10\,S_c/H - (H + 3d)]\,(H + 3B)^2 \cdots\cdots \quad (25a)$$

CASE III. Surcharged Footing. Fig. 4-I (c).—Likewise:

$$\Delta P_{NET} = \frac{B^2}{(h + B)^2} \quad (p_c - \gamma\, D) + \gamma\, d \cdots\cdots\cdots \quad (27a)$$

$$[h_s + 11D - 10d]\,(h_s + B)^2 - 10D\,h_s\,(h_s + 2B) = \frac{10\,B^2}{\gamma} \quad p_c \cdots \quad (28a)$$

$$\Delta P_{NET} = \frac{9B^2}{(h_s + 3B)^2} \quad (p_c - \gamma\, D) + \gamma\, d \cdots\cdots\cdots \quad (29a)$$

$$p'_c = \frac{\gamma}{27B^2}\,[(H + 3D) \cdot 10^{S_c/H} - (H + 3d)]\,(H + 3B)^2$$

$$- \frac{\gamma}{9\,B^2}\,D\,H\,(H + 6\,B) \cdots\cdots\cdots\cdots\cdots \quad (30a)$$

The methods for obtaining solutions to the above equations for depth of significant stress and limiting contact pressure will follow the author's description in his paper.

ALFRED C. SCHEER,[1] A.M. ASCE.—Mr. Hough has proposed that one-dimensional consolidation theory be used to estimate the settlement of surface footings resting directly on top of a compressible stratum, and strongly implies that the settlement caused by lateral displacement of soil is negligible if the load intensity is less than about half the ultimate intensity.

This contradicts the widely held view that, for surface footings, even small load intensities may cause considerable lateral displacement of soil near the ground surface. A number of prominent authors have exhibited a marked reluctance to advocate the computation of footing contact settlement from one-dimensional consolidation data. The prestige of the plate-bearing test, as a tool for estimating settlement, rests in part on the belief that settlement due to lateral displacement is more than negligible.

Perhaps Mr. Hough's position is defensible on purely pragmatic grounds, inasmuch as observed settlements of foundations have been rather consistently less than predicted. On the other hand, most of the case histories cited in the literature are for deep foundations.

Mr. Hough would enhance the value of his paper if he would offer a convincing defense of his thesis that the settlement of a surface footing is, in fact, due primarily to one-dimensional consolidation when the footing rests directly on the compressible stratum.

---

[a] August, 1959, by B. K. Hough.

[1] Assoc. Prof. of Civ. Engrg., Montana State College, Bozeman, Mont.

INVESTIGATION OF UNDERSEEPAGE - MISSISSIPPI RIVER LEVEES[a]

Discussion by Max Suter

MAX SUTER,[1] F. ASCE.—This discussion has been written by the author in his retirement, far from the office in which he did the original work, and the statements are his responsibility.

The paper assumes that seepage and sand boils landward of Mississippi River levees are due to seepage under the levee from the river. On this assumption as to cause, it attempts to explain certain flow conditions and discusses certain remedial measure.

Now, what are the reasons on which this assumption is based?

1. There is a great head during floods from the river side to the land side. This makes it look evident that flow from the river to the land side must occur under the levee.

However, looks can be deceiving; it looks, also, that the sun revolves around the earth.

2. A gradient has been established across the levee. From the data given, at most two points of this gradient have been established in the levee, and a straight-line extrapolation made. From measurements made by the Army Engineers in Peoria, Illinois, but never published, on a series of well points across the levee, it is known that a very steep curved gradient exists within the levee which cannot be extrapolated. It may be that on the lower Mississippi levees other conditions exist, but this has not been shown.

3. Geological studies show greater seepage at places with high horizontal permeability. This permeability does not indicate the direction of flow and a variation in flow can always be expected with a variation in permeability, other conditions remaining the same.

4. It is assumed that all the seepage flow and pumpage flow is from the river and this is based on a fair agreement between measured flows and calculated flows. However, considering the assumptions made in the calculations, agreement can be reached.

The fundamental assumption on which this paper is based is therefore on a weak foundation.

The paper itself gives some indication thereof by admitting that "no crevasses of Lower Mississippi levees have been positively attributed to sand boils or piping since 1913."

---

[a] August, 1959, by W. J. Turnbull and C. I. Mansur.

[1] Prin. Engr., Emeritus, Illinois State Water Survey, now in San Clemente, Calif.

The writer has had many occasions to study sand boils in Illinois, from Peoria to Cairo, for the State Water Survey. In these studies the conclusion was reached that most sand boils are not due to seepage from the river side, but due to blocking the outlet of the natural groundwater into the stream during floods.

The evidence for this conclusion merits short listing:

a. No levee failure, not even the slightest settling of a levee, was observed near sand boils.

b. Sand boils carried tons of sand to the surface, but in every case where settling could be established it occurred on the land side of the sand boil.

c. The flow in sand boils can be stopped by sandbagging for a relatively small height and far below the river stage. If all flow could be stopped and the water comes from the river, it would reach hydrostatic river level.

d. Temperature measurements made on river water, sand boil water, and in a nearby well, show that the temperature of the sand boil water is close to that of the well water, and differs from that of the river water, provided the flood occurs at a season when there is a difference of temperature between river and well water. Water has the highest specific heat of any substance and does not change its temperature without a considerable exchange of energy. The dropping of the water from river stage to landside ground does not furnish this energy, as water falling 100 ft is raised in temperature only 0.13° F if all the energy of the fall is transferred into heat. And anyhow, if this dropping would furnish the energy for warming the water in winter, it could not also act for cooling the water in summer.

e. Chemical analysis of three waters from locations as described for temperature show also that the sand boil water is ground water rather than river water. Many of the chemical analyses made in these investigations have been published by the State Water Survey in "The Floods of May 1943 in Illinois," issued by the Division of Waterways, 1943.

These data are not here reproduced, as the main purpose of this discussion is to show that the fundamental assumption on which the paper is based appears to be untenable in most cases. If so, then many of the calculations in the paper are merely mental gymnastics. Even the control measures have to be judged from a different standpoint because the total flow is not all flow from the river, as assumed in the paper, but mainly flow from the hills. This effects the interpretation of the action of relief wells, cutoff walls, berms, borrow pits, etc. However, it is announced that control measures will be treated in another paper and are therefore not discussed here.

The aim of this discussion is to show that another interpretation to the problem is needed than that given in this paper. All factors have to be taken into consideration, not just those that fit a superficially "evident" explanation.

It is suggested that during future flood investigation of sand boils, attention is also given to temperatures and the chemical composition of the water, because nature does not neglect these factors.

ROCK CHARACTERISTICS OF THE PAUL AFONSO POWER PLANT[a]

Discussion by A. A. Eremin

A. A. EREMIN,[1] M. ASCE.—In 1878, Albert Heim, Swiss Geologist, stated that the rock pressures in the ground follow the hydrostatic rules. Numerous tests have been performed since that time to determine the limitations of Heim's rules.

In this paper the authors verified the fact that the vertical residual stresses in the tunnel bores are greater than the horizontal stresses and thus confirmed Heim's rules. From the tests the authors concluded that the modulus of elasticity of the rocks obtained in the laboratory tests are greater than that in the tests of the rocks in situ. It should also be considered that the rock tests in the laboratory are in the unconfined conditions while the tests of rocks in place were affected by distribution of the compressive strain in the ground.

The authors attributed the lower modulus of elasticity of the rocks in the tunnel bores to the effect of rock intrusions and cracks in the rocks. It is regreted that the tests of the rocks were not performed in rocks without intrusions and cracks. Obviously, such tests would give interesting numerical information on the effect of cracks and intrusions on the elastic properties of the rocks.

The literature on the elastic properties of rocks is limited. Therefore, the authors should be congratulated for their useful contribution.

[a] August, 1959, by Ernesto Pichlar and Franciso Barros de Campos.

[1] Assoc. Bridge Engr., Bridge Dept. Calif. State Highways, Sacramento, Calif.

CONSTRUCTION MATERIALS CONTROL—AASHO ROAD TEST[a]

Discussion by Edward A. Abdun-Nur

EDWARD A. ABDUN-NUR,[1] F. ASCE.—The author is to be commended for presenting to the profession the very ingenious and successful methods used to control materials on the AASHO Road Test. The close tolerances required on this project, and the speed in making decisions and approving work, are indications of what can be done to control construction operations without delay. No longer valid is the excuse that control will slow up construction.

The writer applied with very excellent success similar statistical approaches to control some construction phases of the Illinois Toll Highway, which was built at essentially the same time as the AASHO Road Test. The Toll Highway differed from the Road Test in that it was not set up as a research project where the contractor would expect novel practices, but rather as a fast construction job, speeded up under the pressure of about $1,500,000 per month interest on the bonds. Because it involved 23 engineering firms and some 60 general contracts running simultaneously, it was imperative not only to develop methods to realize the quality control desired, but to do it in such a manner that no delays could be attributed to the control procedures.

Essentially, the method used was similar to that described in Fig. 2, except that the procedure described by the author was reversed. The "out" limits that would be acceptable were clearly defined in the specifications, so that the contractor knew what leeway he had. From these acceptable "out" limits and any given coefficient of variation for an individual contractor operation (this coefficient remains reasonably constant for a given crew and equipment on given job conditions), the average that the contractor had to attain to remain within the specifications could then be calculated, and the contractor would know what he had to work towards. This average could be checked at any time with very simple arithmetic to determine compliance.

Naturally, the contractor tried to remain a little above the average required for his operation, and thus well within the "out" limits. By arriving at the required average in this manner and keying it to the degree of uniformity attained in the operation, the contractor was provided with a sliding scale requirement that was in effect an incentive—the lower his coefficient of variation (the more uniform his operation), the lower the required average, and the lower the latter the more profit he could make; the higher the coefficient of variation attained in his operation, the higher the required average, and therefore the less profit. In this manner, the contractor strove towards uniformity, and thus towards better control. This, in effect, placed him on the same side

---

[a] October, 1959, by James F. Shook.

[1] Cons. Engr., Denver, Colo.

of the fence as the engineer. It induced cooperation regarding quality instead of arguments.

It is believed that as applied on the Illinois Toll Highway, this method provided not only an easy way to control operations, but the incentive so badly needed to get the contractor to want quality.

It is most interesting to learn about the results obtained with the nuclear density equipment and the speed with which results were obtained. As nearly as the writer has been able to ascertain, the application of this equipment is limited to thin layers of 3 to 4 in. in depth. It would be of interest to have the author's experience with this equipment on thicker lifts, which are frequently used in highway work. In the case of the Toll Highway, some granular layers were 10 to 11 in. in thickness.

RATE OF CONSTRUCTING EMBANKMENTS ON SOFT FOUNDATION SOILS[a]

Discussion by W. H. Gotolski

W. H. GOTOLSKI,[1] M. ASCE.—Mr. Lobdell's paper provides an interesting approach to the establishment of a method for scheduling filling operations on soft foundation soils. His paper demonstrates the procedures required to develop his proposed analysis and reveals the lack of information or methods available for computing the per cent consolidation at a given depth in a clay layer for the three-dimensional case where sand drains are involved.

Mr. Lobdell mentions pore pressure and height of fill relations several times. One method which appears to work satisfactorily is that of determining the allowable pore pressure through an analysis of the stability of the embankment at various stages of construction. The Swedish Circle method may be used and the work facilitated by use of electronic computers. The method would make use of locating the centers (designated by "O") of circles along the mid-slope line and summating moments about these points. The safety factor, FS, is equal to the ratio of the sum of the resisting moments to the sum of the driving moments. If the safety factor is greater than one for all positions of "O" along the mid-slope line the embankment may generally be considered stable.

The primary variable is the strength of the underlying soil, and this depends on pore water pressure, as pointed out by Mr. Lobdell, and the soil density. During the consolidation tests in the laboratory, pore water pressures can be measured under various applied loads and at various degrees of consolidation. Then shear tests can be performed with the same applied loads and degrees of consolidation. Thus a relationship between pore-water pressure and the strength may be obtained. For any given height of embankment the strength of underlying soil necessary for stability is computed. In the field, the pore-water pressure is measured and kept below the point where it would reduce the shearing strength of the soil to a dangerously low value. The allowable pore pressure varies with the height of the embankment; the higher the embankment the lower the allowable pore pressure.

The pore pressure devices should be read daily. The rate of placing the fill is then controlled by maintaining the pore water pressure at a predetermined value.

The writer believes R. A. Barron[2] has presented a method which can be used to compute the per cent consolidation at a given depth in a clay layer for

---

[a] October, 1959, by Herbert L. Lobdell.

[1] Dept. of Civ. Engrg., Pennsylvania State Univ., Univ. Park, Pa.

[2] "Consolidation of Fine-Grained Soils by Drain Wells," by R. A. Barron, Trans. ASCE, Vol. 113, 1948, pp. 718-754.

the three-dimensional case where sand drains are involved. Mr. Barron included in his paper a diagram similar to Fig. 4 of Mr. Lobdell's paper. This is the figure showing consolidation as a function of depth and time factor for vertical flow. Barron indicates a method[2] that may be used for determining the excess pressure at a point in a clay layer. This is accomplished by making use of Fig. 4 (Proc. Sep. ASCE, No. 2214, 1959) and a figure relating radial consolidation rates at various radial surfaces for a clay cylinder.[2] This latter figure relates u_r, the excess pore-water pressure due to radial flow, and T_h, the time factor for horizontal drainage. A numerical example of this nature may be found in Mr. Barron's paper.

All that has been discussed here has undoubtedly been considered by others, but the writer is of the opinion that it will do no harm to bring it to the fore. He is also aware of the fact that some of these statements have been expressed before by other authors, but he feels that these are meant only as suggestions and not as criticism.

In this discussion the writer does not wish in any way to detract from the value of the author's suggested procedure. The author's paper establishes a firm basis for determination of the construction rate of embankments on soft foundation soils.

MAJOR POWER STATION FOUNDATION IN BROKEN LIMESTONE[a]

Discussion by Ralph B. Peck

RALPH B. PECK,[1] F. ASCE.—It may be of interest to examine in greater detail the reasons for the use of two basically different types of foundations beneath two parts of the same power plant. As the authors state, the foundation conditions for Unit 3 were significantly poorer than those for Units 1 and 2. The nature of the difference was rather striking.

The raft foundation for Units 1 and 2 was about 160 ft by 240 ft in plan and exerted an average unit load at its base of about 6000 lb per sq ft. The superstructure contained 84 columns. At almost every column a rock boring was made; the total of 88 borings included a few to explore intermediate locations. The boring program was under the close supervision of the consulting geologist who recognized the sink-hole character of the colomite, as exemplified by the authors' Fig. 2, and the collapses associated with the weight of the glaciers. On the basis of the appearance of the cores, the amount of recovery, the drilling time and water loss, the geologist categorized the materials as (1) sound rock, (2) slightly fractured rock, or (3) weathered or badly fractured rock with clay seams, shales, clays, open cavities, or construction fill.

In considering a raft foundation, principal attention was devoted to the relative amounts and distribution of areas of support or lack of support. In this connection, judgment was required to define the area surrounding a boring that might properly be considered as represented by the boring. In part this judgment was aided by a few borings at close spacing, but principally by examination of exposed faces in neighboring quarries where sinkholes were prevalent in the same formation. It was concluded that positive inferences could not be drawn at distances greater than about 3 ft from a boring, but that a reasonable statistical picture could be obtained from the 88 borings.

The results of the borings were plotted as a series of longitudinal and cross sections, of which two along the west side of the building, Profiles B and C, are shown in Fig. 1. The heavy horizontal line in each profile is the proposed level of the base of the mat. It may be inferred that fairly substantial support was provided along the central part of Profile C, but that several sections along Profile B were underlain by zones of poor rock support. However, for the design of the mat the distribution of support had to be known in plan. Hence, the results of the profiles were combined into a summary diagram, Fig. 2. The diagram is necessarily idealized but it serves a useful purpose for engineering design. It indicates the general dimensions and locations of zones that must be bridged by the raft.

---

[a] October, 1959, by Swiger and Estes.

[1] Prof. of Foundation Engrg., Univ. of Illinois, Urbana, Ill.

On the basis of Fig. 2 it was concluded that the principal areas where support was lacking were at the two ends and along the east side of the structure, closest to the canal. Nevertheless, even in these localities the minimum width of the unsupported area, representing the spaces that would have to be

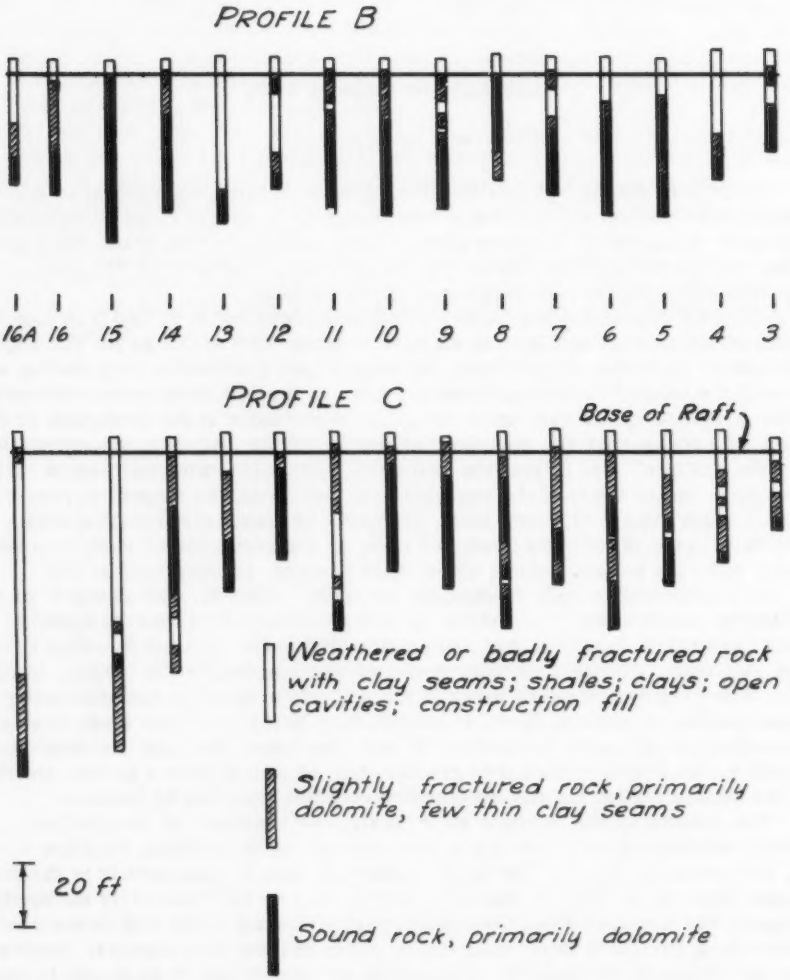

PROFILE B
16A 16 15 14 13 12 11 10 9 8 7 6 5 4 3
PROFILE C
Base of Raft
Weathered or badly fractured rock with clay seams; shales; clays; open cavities; construction fill
Slightly fractured rock, primarily dolomite, few thin clay seams
20 ft
Sound rock, primarily dolomite

FIG. 1.—LONGITUDINAL SECTIONS

bridged by the raft, was generally less than 30 ft. In most localities there would be occasional areas of good support that would lead to slab action in two directions and would appreciably reduce the stresses in the slab.

The diagram also indicated two areas that deserved particular attention. One of these was the corner of the building between column lines F and G,

and 14 and 16A. Here the spans were relatively large and it appeared as if it might be necessary to construct a support near the intersection of lines F and 15 where the depth to sound material was on the order of only 10 ft. The other was along line G between column lines 8 and 6 where the slab might be required to carry its load by cantilever action for a distance of about 40 ft.

A careful study of the major column loads established the fortunate fact that the heaviest loads occurred generally where the support was relatively

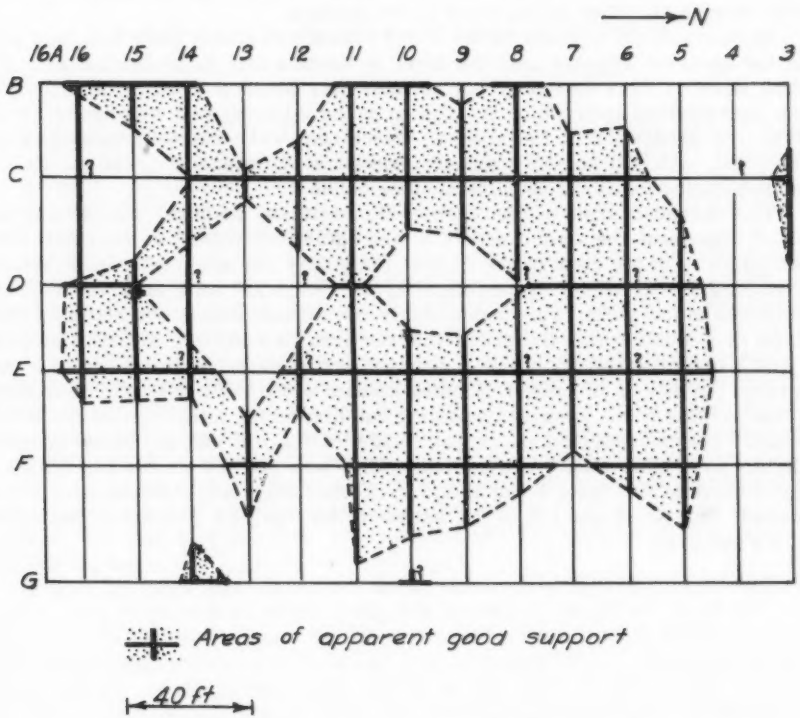


FIG. 2.—SUMMARY OF SUBGRADE SUPPORT AT BASE OF RAFT

The raft foundation was selected because it was felt that, on account of the variations in compressibility of the rock in all parts of the deposit, the most uniform support and the least differential settlement would be experienced if the average load per unit area at all points were kept reasonably small, and if the structure itself possessed considerable rigidity. It was considered undesirable to deliver a high concentration of load to the rock at any point, as by means of piers or shallow caissons, inasmuch as the likelihood of a small and undetected zone of poor support at a given point was relatively high. Since the structural considerations indicated that the raft was feasible, this type of foundation was adopted.

The structural investigation of the raft was based on the assumption that no upward reaction could be developed beneath the zones of poor support,

whereas the entire weight of the superstructure would be distributed uniformly over the zones of adequate support. In addition the raft was designed to carry the downward loads from the superstructure even if there were an isolated zone of inadequate support with a diameter of 20 ft at any point.

The raft, designed on these premises, was constructed. Settlement was too small to be detected by ordinary leveling techniques and the foundation proved entirely satisfactory. Moreover, excavation for the raft disclosed conditions compatible with the assumptions. The rock surface was typical of the sinkhole terrain expected on the basis of the borings.

In contrast, the exploration for Unit 3 indicated several large and deep gaps in the apparent support near the level at which a raft might be founded. The gaps were at first disconcerting because they seemed to be at variance with the assumptions upon which the design of the adjacent raft was based. However, the excavation of the area, as described by the authors, suggested that the gaps, although possibly merely examples of unusually great subsurface solution along joints, might have another cause.

The unshaded area in the Turbine Room, in the authors' Fig. 4, may instead represent the upper end of a short gulley tributary to a former main drainageway to the right of the figure. The gulley was apparently being eroded headward by a set of waterfalls. At the extreme head of several of the branches of the gulley were found nicely preserved potholes or kettles. Thus there is a considerable likelihood that the area immediately adjacent to Units 1 and 2 had experienced not only solution and collapse, but subsequently had become the site of an actively eroding stream which cut a deep gash beginning in the turbine room area and widening out toward its outlet beyond the boiler house. The unsupported corner at G-16A of the old raft probably extended slightly over this area. Although the gulley was not in a position to render a raft infeasible for Units 1 and 2, and it created very large unsupported areas beneath Unit No. 3 and led to the necessity for the pier foundation described by the authors.

DESIGN OF UNDERSEEPAGE CONTROL MEASURES FOR DAMS AND LEVEES[a]

Discussion by Marcel Bitoun and Jorgen Christiansen

MARCEL BITOUN,[1] M. ASCE and JORGEN CHRISTIANSEN[2].—The following is meant as a modest contribution to the problem so competently described by Messrs. Turnbull and Mansur. It deals with the case of a small size levee where underseepage control was complicated by the presence of a groundwater table at a short distance beneath the levee.

The Problem.—In 1949 the Pennsylvania Department of Forests and Waters constructed a levee over 15 ft in height along the Susquehanna River and Shephards Creek at their confluence, as part of the Sayre Flood Protection Project. During stripping and excavation operations, a gravel layer was discovered in the foundation of the levee. Construction was temporarily stopped while a subsurface investigation was conducted by means of auger borings at a spacing of 200 ft to 400 ft along the center line of the levee. Approximate boundaries of the gravel zone were thus established and a report prepared, describing various methods of sealing the pervious zone. However, no measures were taken at that time to prevent seepage under the levee; probably because excavation of a trench and backfilling with rolled impervious material was not economically feasible; other solutions could be studied after completion of the levee.

It is to be noted that the borrow area, which provided impervious material used in the construction of the levee, was located on the riverside of the levee between the Lockhart Street Bridge and approximate Station 40+00 (Fig. 1). The overlaying impervious material was stripped off to a distance of 40 ft from the toe of the levee; the gravel zone was exposed and the seepage path shortened by roughly 350 ft. This, of course, aggravated the conditions, as anticipated.

A deep hole in the original ground located approximately 60 ft from the riverside toe of the levee near Station 35+30 was, however, filled to grade with spoil material, while the project was being completed.

Observation of Seepage.—Shortly after completion of the levee, ponding of water occurred in the low areas behind the levee when the river reached moderate flood stage. This occurred thereafter with a frequency of about once every 2 yr. Underseepage water always appeared first, directly across the levee from the filled hole mentioned above. Local residents reported that water and air bubbles were forced out of the ground shortly after the river

---

[a] October, 1959, by W. J. Turnbull and C. I. Mansur.

[1] Chf. Design Branch, Div. of Flood Control, Pa. Dept. of Forests and Waters, Harrisburg, Pa.

[2] Hydr. Engr., Div. of Flood Control, Pa. Dept. of Forests and Waters, Harrisburg, Pa.

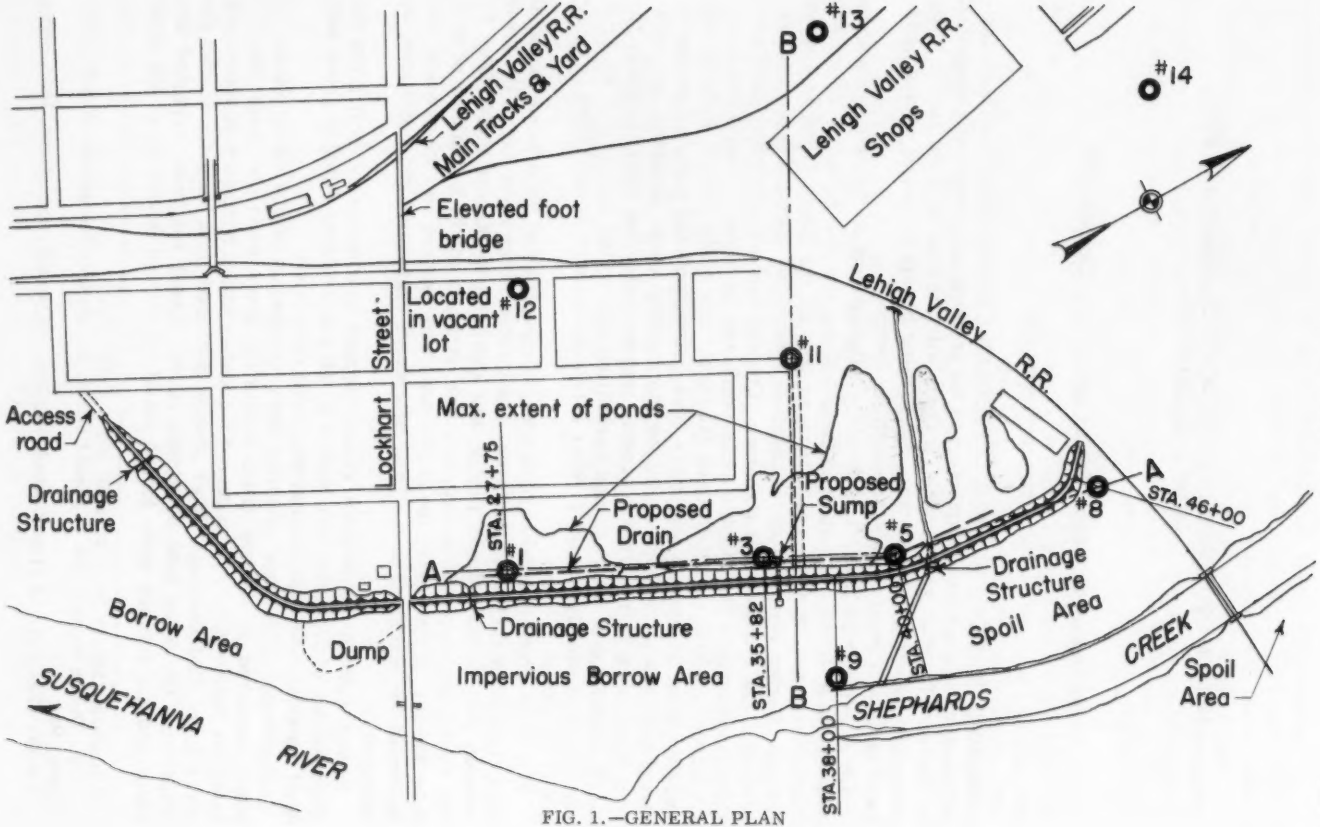


FIG. 1.—GENERAL PLAN

stage reached the elevation of the banks. The ponding water kept rising quickly until a short time after peak stage was reached, and the low areas remained flooded for approximately a week after the river had returned to low stage. Attempts by residents to lower the water level in the pond with fire pumps were unsuccessful. Twelve dwellings were affected by this recurrent flooding and the situation was quite serious.

It is worth mentioning here that reliable reports indicate that filling of the low areas became progressively more rapid each time resurgence of underseepage occurred. Though no definite piping was observed by engineers, this was interpreted as an indication of migration of fines in the subsoil. The Department became increasingly concerned, for a blowout would have extremely grave consequences. Corrective measures were thus imperative.

Subsurface Investigation.—The information obtained from the auger borings made during construction of the levee was very summary and incomplete. In particular, no record of the groundwater table had been kept. In order to properly design the corrective measures, it was decided to carry out an extensive program of investigations.

The purpose of this program was to determine, as well as possible, the boundaries of the pervious zone, the nature of the material above and below it; the permeability characteristics of the various strata involved; the elevation of the groundwater table, and to record its seasonal fluctuations. The original schedule included drive sample borings, auger borings, installation of piezometers, inspection pits, field and laboratory soil tests.

The major portion of this work was performed during the month of December 1958 under the supervision of the second writer.

This investigation confirmed the presence of a highly permeable layer of gravel under the general area.

The logs generally showed the following:

a) 1 ft topsoil;
b) 1 ft to 6 ft of a fine brown sandy silt;
c) The pervious layer extends from there down to 10 ft to 14 ft below the ground; and
d) At this depth, the soil changes to a very fine gray sandy silt and/or the gravelly soil becomes so rich in fines that its permeability is not greater than that of the sandy silt.

The groundwater table was located and the nine standpipes (Fig. 2) installed to record its fluctuations. The piezometers are made of perforated 2-in. standard steel pipes, capped and padlocked. The space between the observation pipe and the wall of the boring hole was filled with screened and washed gravel upon withdrawal of the casing. The top 3 ft were plugged with mortar. As had been suspected, a relatively large amount of water flows underground toward the river. Fig. 3 shows profiles along two perpendicular directions.

Results of Field and Laboratory Testing.—

a) Recovery of samples from borings: Continuous sampling was made in the drive-sample borings. Auger borings were sampled approximately every 5 ft. Below the groundwater level, however, recovery of the gravel samples was quite difficult. The number of flows per foot of penetration of a standard 2-in. spoon sampler was recorded, which indicated that the granular soils in place were loose to medium loose deposits. The blow counts were as listed in Table 1.

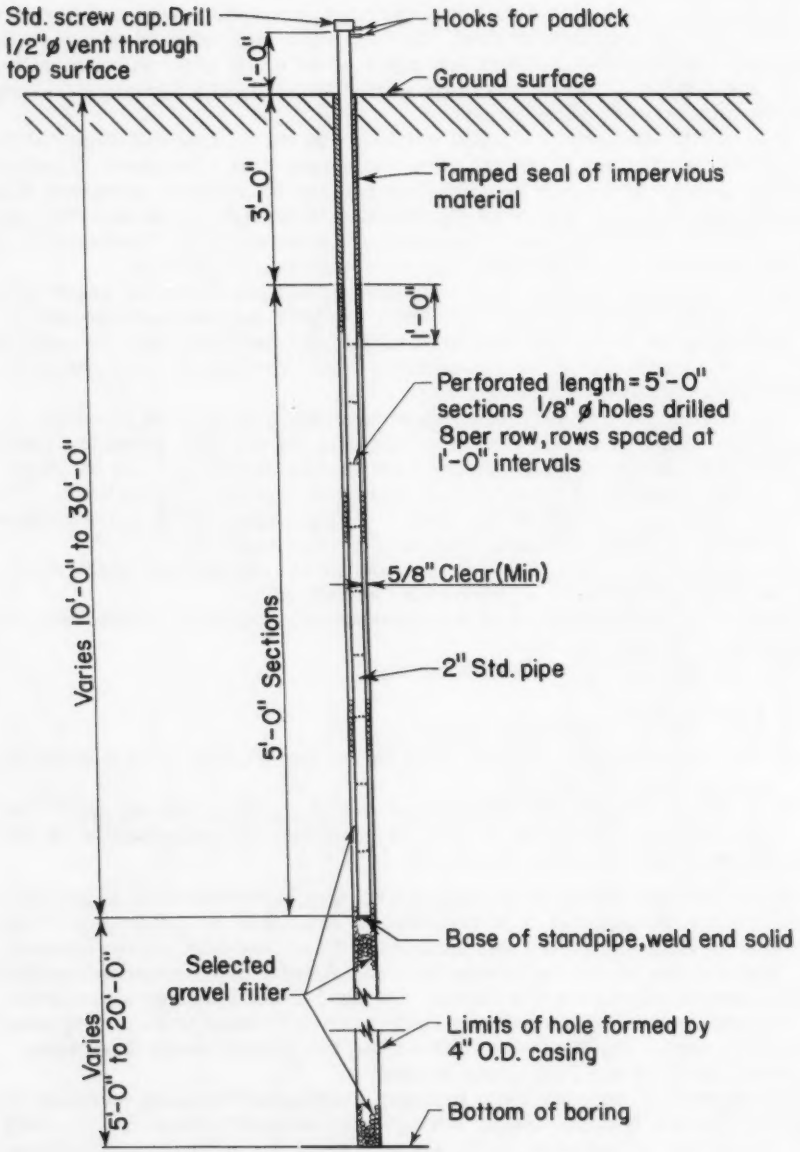
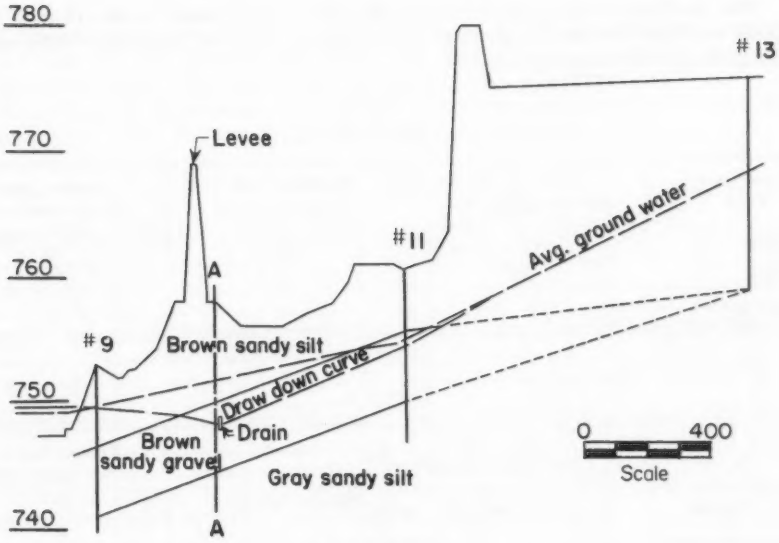
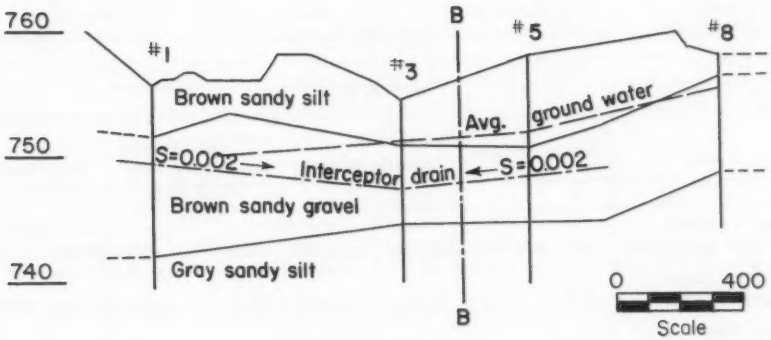

Std. screw cap. Drill 1/2"ø vent through top surface
Hooks for padlock
1'-0"
Ground surface
3'-0"
Tamped seal of impervious material
1'-0"
Perforated length = 5'-0" sections 1/8"ø holes drilled 8 per row, rows spaced at 1'-0" intervals
Varies 10'-0" to 30'-0"
5'-0" Sections
5/8" Clear (Min)
2" Std. pipe
Base of standpipe, weld end solid
Selected gravel filter
Varies 5'-0" to 20'-0"
Limits of hole formed by 4" O.D. casing
Bottom of boring

FIG. 2.—TYPICAL DETAIL OF PIEZOMETER



AXIS B-B



AXIS A-A

FIG. 3.—PROFILES

b) Field permeability testing: The method used is that described as "Field Permeability Test B" on page 237 of the U. S. Bureau of Reclamation Earth Manual.

The section tested was the hole left after withdrawal of the 18 inch long spoon sampler below the casing. The results of these tests were consistent enough to be considered reliable.

TABLE 1

Blows per Foot	Upper Layer (Brown sandy silt)	Pervious Layer (Sandy Gravel)	Lower Layer (Gray sandy silt)
Max.	13	76	32
Min.	3	6	2
Average	7	21	9

TABLE 2

K, in centimeters per second	Upper Layer (Brown sandy silt)	Pervious Layer (Sandy Gravel)	Lower Layer (Gray sandy silt)
		Number of tests	
	7	11	9
Maximum	1.1×10^{-5}	1.4×10^{-1}	1.2×10^{-4}
Minimum	$< 10^{-8}$	3.9×10^{-3}	3.5×10^{-6}
Average	6.2×10^{-5}	4.4×10^{-2}	6.0×10^{-5}

TABLE 3

Layer	Water Content, in %		Specific Gravity		Dry Unit Weight, in Pounds per Cubic Foot		Void Ratio	
Upper Layer (Brown sandy silt)	34.7	26.0	2.71	2.76	86.0	99.6	0.965	0.730
Pervious Layer (Sandy Gravel)	7.2	7.1	2.46	2.46	127.0	122.8	0.222	0.250
Lower Layer (Gray sandy silt)	26.7		2.71		...		...	

The computed coefficients of permeability K (cm/sec) are given in Table 2.

c) Test Pits: Pits were opened to determine the natural moisture and density in-situ of the brown sandy silt and the sandy gravel. In-place determinations were made, and in addition, undisturbed block samples were taken and tested in the laboratory. The results are presented in Table 3.

d) Laboratory Testing: Approximately thirty samples were tested for the purpose of classification according to the "Unified Classification System."

The upper and lower silts are very poorly graded and classify as ML.

The pervious zone consists of a well graded gravel-GW.

Fig. 4 shows typical gradation curves representative of the materials in place.

Correlation between River Stage and Groundwater Table.—Daily gage-height data for the Susquehanna River near Sayre were received through the courtesy of the U. S. Geological Survey Branch Office in Ithaca, N. Y.

Periodical readings of the groundwater table elevation in the nine (9) piezometers were made through cooperation of local residents.

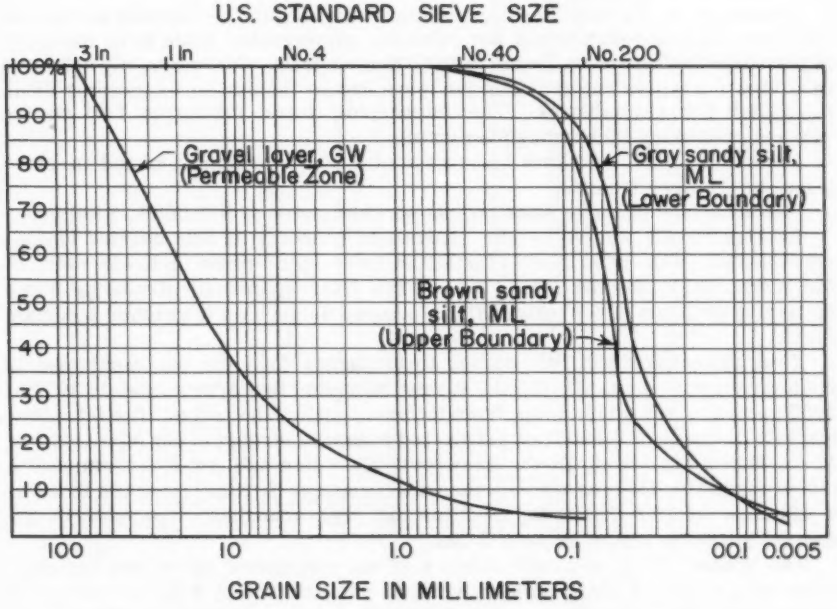


FIG. 4.—TYPICAL GRADATION CURVES OF LAYERS ENCOUNTERED

Fig. 3 shows that the groundwater table within 600 ft of the levee is influenced by the fluctuations of the river stage. Beyond this distance (approximate location of piezometer No. 11), the groundwater table follows more directly the amount of rainfall percolation.

Groundwater contour maps were drawn for maximum, minimum and average values of the standpipe readings, and gradients of the water surface were determined for each condition. Corresponding values of the discharge of the groundwater table were computed by application of Darcy's formula, assuming that all of the flow occurred in the gravel zone. The relative values of permeabilities justified this assumption.

It was thus found that a quantity of 70 to 190 gpd per lin ft flow underground toward the river.

It is obvious that a cutoff under the levee would not have been a satisfactory solution, for the water table in the area would have been raised.

Moreover, it is also evident that the groundwater table contributed to a certain extent to the interior flooding during high stages.

For design purposes, it had been computed that a maximum discharge of 200 gpd per lin foot should be taken into account for the section between the Lockhart Street Bridge and Station 35+30, and 150 gpd per lin ft in the upstream section, to Station 43+00.

Review of the Various Possible Solutions Investigated.—The only acceptable solution was one which would not raise the groundwater table to an elevation higher than the basement floors in the affected area. Therefore, all solutions involving a positive cutoff under the levee had to include an interceptor drain to collect the groundwater. The interceptor would discharge into one or several collectors laid through the cutoff.

The cutoff itself could have been made of either overlapping mixed-in-place piles, or a slurry trench.

The total cost of these installations was estimated at $75,000 to $100,000.

Scheme Finally Adopted.—For reasons of economy, it was decided to omit construction of a cutoff and to install an interceptor drain on the landside of the levee, deep into the gravel zone. The interceptor will discharge into a collecting pit from which water will be pumped as soon as it reaches a certain level.

This interceptor had, of course, to be larger than the one mentioned in conjunction with a cutoff. Indeed, it was computed that approximately 1000 to 1400 gpd per lin ft would seep from the river under the levee. The interceptor will be made of 1,595 ft of perforated standard strength clay pipe (1445 ft, 12-in. diameter; 150 ft, 15-in. diameter). This line will amply accommodate the groundwater flow as well as the underseepage. It runs along the levee at a distance of 10 ft from the land toe and both branches slope towards the natural low point with a slope of 0.002.

The drained water will collect into a 96-in. corrugated metal pipe installed vertically, on top of which an 8-in. vertical pump driven by a 20 hp motor will be installed (Fig. 5). The design discharge of the pumping installation is 1500 gpm. However, provision has been made for the installation of a second pumping unit, would it become necessary in the future. The pump will discharge into an 8-in. steel line laid over the top of the levee, to a flapgated outlet structure. Operation of the pump will be automatically controlled by a float working between two predetermined elevations.

Construction of the whole installation is estimated to cost approximately $30,000.

Studies of the flow duration curve for the Susquehanna River show that the pumping installation will be in operation 1.5% of the year, or approximately 135 hr. The annual power consumption would amount to about $150.00.

Acknowledgement.—Presentation of the foregoing date is made with the permission of the Pennsylvania Department of Forests and Waters. Several staff members, in particular Van A. Silver, contributed to the field investigations and the design of the Sayre Underseepage Control System.

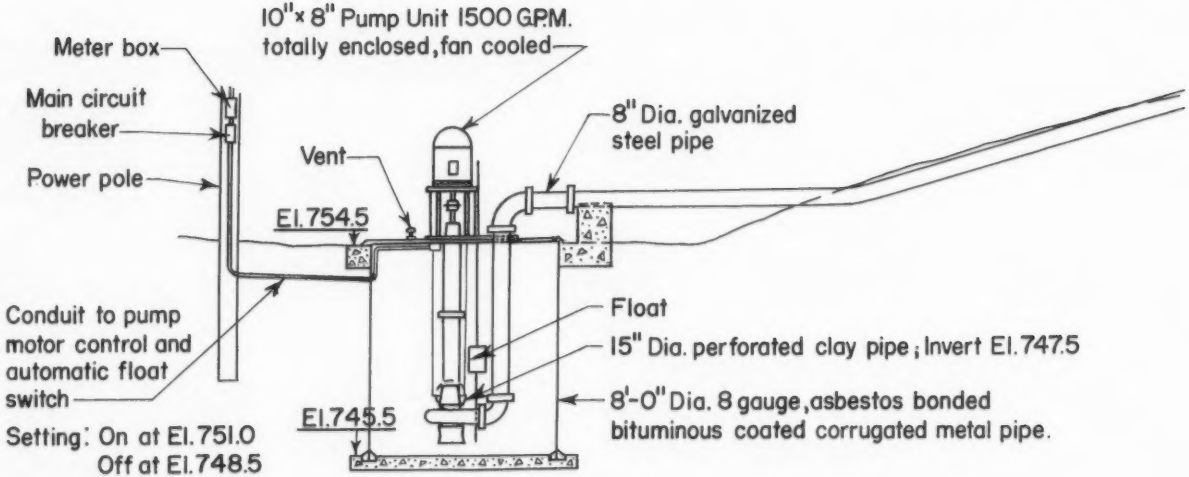

10"× 8" Pump Unit 1500 G.P.M.
totally enclosed, fan cooled
Meter box
Main circuit breaker
Power pole
Vent
8" Dia. galvanized steel pipe
El. 754.5
Conduit to pump motor control and automatic float switch
Setting: On at El. 751.0
Off at El. 748.5
El. 745.5
Float
15" Dia. perforated clay pipe; Invert El. 747.5
8'-0" Dia. 8 gauge, asbestos bonded bituminous coated corrugated metal pipe.

FIG. 5.—PUMPING INSTALLATION

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1),1956(CO1),1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c]. 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3),2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3),2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3),2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4),2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1),2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6),2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

VOLUME 86 (1960)

JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

c. Discussion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1960

PRESIDENT

FRANK A. MARSTON

VICE-PRESIDENTS

Term expires October, 1960:
PAUL L. HOLLAND
LLOYD D. KNAPP

Term expires October, 1961:
CHARLES B. MOLINEAUX
LAWRENCE A. ELSENER

DIRECTORS

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

Term expires October, 1961:
THOMAS J. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIUS
WAYNE G. O'HARRA
FRED H. RHODES, JR.
N. T. VEATCH

Term expires October, 1962:
ELMER K. TIMBY
SAMUEL S. BAXTER
THOMAS M. NILES
TRENT R. DAMES
WOODROW W. BAKER
BERNHARD DORNBLATT

PAST PRESIDENTS

Members of the Board

LOUIS R. HOWSON
FRANCIS S. FRIEL

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
E. LAWRENCE CHANDLER

ASSISTANT SECRETARY
E. LAWRENCE CHANDLER

ASSISTANT TREASURER
ENOCH R. NEEDLES

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

MARVIN L. SCHECHTER
Associate Editor of Technical Publications

IRVIN J. SCHWARTZ
Assistant Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

PHILIP C. RUTLEDGE, *Chairman*

THOMAS M. NILES, *Vice-Chairman*

TILTON E. SHELBURNE
WESTON S. EVANS
WAYNE G. O'HARRA
BERNHARD DORNBLATT

PART 2

FEBRUARY 1960 — 6 NO. SM 1
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

February, 1960

THE KARL TERZAGHI AWARD

The Soil Mechanics and Foundations Division is planning an award to honor Professor Karl Terzaghi, Honorary Member of ASCE, and the Father of Modern Soil Mechanics. He is respected throughout the world for establishing the science of soil mechanics, and for his many contributions to theoretical and practical aspects of earthwork and foundation engineering.

An unsolicited editorial which appeared in the October 1, 1959 issue of Engineering News-Record expresses admirably our objectives; it is reproduced below, with permission of the publishers:

A KARL TERZAGHI AWARD

"THE ENGINEERING PROFESSION has an unusual opportunity to honor itself by supporting a proposal to establish an award honoring Karl Terzaghi, the Father of Soil Mechanics and a famed foundation consultant. From earlier times, names of notable engineers have been handed down to become a part of the civil engineer's language-the Whipple truss, MacAdam roads, the Francis turbine, the Imhoff tank, to mention a few. No name of modern currency could be more appropriately memorialized than that of Terzaghi, in connection with the science of soil mechanics, which he established and has continued to serve with such dedication and success.

"The proposed award is the idea of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers. It would be made for a paper published by the Society, judged to set forth truly exceptional contributions to engineering knowledge in the field of foundations. And this is particularly appropriate since papers by Professor Terzaghi himself have so often been in this category, and, moreover, have been presented verbally with such lucidity and brevity that many of them have had an immediate and profound effect.

"Funds are being sought to establish the Karl Terzaghi Award. Money for stimulating the future development of the profession could not be put to better use."

Note.—No. 1960-6 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, SM 1, February, 1960.
Copyright 1960 by the American Society of Civil Engineers.

We hope and expect that the Karl Terzaghi Award will soon be one of the most respected and valuable awards offered by the Society and urge all engineers interested in earthwork and foundation design or construction to enjoy the personal satisfaction of assisting in its establishment.

It is hoped that all members of the Division will assist in establishing this Award. Contributions should be made out to "Mr. Harold T. Larsen, ASCE " and sent to either of the following:

Professor H. B. Seed
Engineering Materials
Laboratory
University of California
Berkeley, California

Mr. Stanley J. Johnson
Moran, Proctor, Mueser
& Rutledge
415 Madison Avenue
New York 17, New York

TO:
The Terzaghi Award Committee

Gentlemen:

I am pleased to pledge* / enclose my check in* the amount of _____
to assist in establishing the KARL TERZAGHI AWARD.

Name _____

Address _____

City & State _____

*Cross out portion not applicable

EXPANSIVE CLAYS

Golden, Colorado -- A practical discussion of clay terrain construction problems is the topic of the most recent Colorado School of Mines Quarterly. The book is a compilation of papers dealing with the theoretical and practical treatment of expansive clays.

The Quarterly (Vol. 54, No. 4) is an outgrowth of the First Annual Soil Mechanics Conference, held in April at Mines. The papers discuss the problems of building on the expansive soils of the Middlewestern and Southwestern areas of the United States.

The conference's aim was to further develop accumulated knowledge of shrinking and expanding soils in a form that can be readily utilized by engineers and architects. It also has a direct connection to building contractors and real estate planners.

Included in the Quarterly are papers by Raymond E. Means, Professor of architecture, Oklahoma State University; Dr. T. William Lambe, Head of MIT's soil engineering department; Chester McDowell, Supervising soils engineer for the Texas Highway Department; Raymond F. Dawson, Civil Engineering Professor at the University of Texas; and W. G. Holtz, earth laboratory branch chief, U. S. Bureau of Reclamation.

The Quarterly, priced at $2.00, may be obtained by writing the Department of Publications, the Colorado School of Mines, Golden, Colorado.

ALFRED A. RAYMOND AWARD

The following men have received awards for the Year 1959.

PRIZE WINNING PAPER - "Lateral Stability of a Flexible Pier"

By: M. T. Davisson

University of Illinois
Urbana, Illinois

HONORABLE MENTION - "The Challenge of the Peace"

By: G. D. Morrison

Structural Engineering Services Ltd.
Calgary, Alberta

"Foundation Vibrations"

By: F. E. Richart, Jr.

University of Florida
Gainesville, Florida

"Pile Foundations in Permafrost"

By: W. C. Dias

American Brake Shoe Company
Kellogg Division
Rochester, New York

JUDGES

Ralph B. Peck	E. A. Dockstader	William W. Moore
Professor of Foundation Engineering University of Illinois	Consulting Engineer West Newton, Mass.	Partner - Dames & Moore Consulting Engineers San Francisco, Cal.

REPORT ON TEXAS SECTION MEETINGS

A technical session was held by the Soil Mechanics and Foundations Group at the Fall Meeting of the Texas Section in Fort Worth, Texas, on October 1, 2, and 3, 1959. Four papers were presented. These were:

"Pavement Design Procedures in the Fort Worth District of the Texas Highway Department," Justin Bingham, State Highway Department, Fort Worth, Texas.

"Test Embankment for Canyon Dam," Ralph Beene, U.S. Corps of Engineers, Fort Worth District, Fort Worth, Texas.

"Rock Queries," Frank G. Bryant, Consulting Engineer, Austin, Texas.

"Investigation of Liquid Limit Test on Soils," Raymond F. Dawson, Professor of Civil Engineering, University of Texas, Austin, Texas.

The new officers of the Soil Mechanics and Foundations Group are:

Dr. Lymon C. Reese, Chairman; Mr. Raymond Mason, Vice-Chairman; Professor Cecil Smith, Secretary.

NEW RUSSIAN JOURNAL ON SOIL MECHANICS

Volumes 1-3 Number 1 of a Russian Journal on Soil Mechanics has come to the attention of the Executive Committee. The name and table of contents has been translated:

Publisher: Scientific-Technical Magazine
Soviet Cabinet Council
Committee for Building Affairs

Magazine Title: Foundations and Soil Mechanics

Contents:

More important problems of foundation construction and 7-year economic program of the Soviet Union.

Use of combined application of the theories of elasticity and plasticity computations to design of foundations.

Computations of foundation design on the basis of limited deformations of a vibro pile driver. (In discussion)

Building and equipment foundations in the west-Siberian Metallurgical Plant.

Moving of two buildings in Moscow.

Experience with sinking a large caisson.

Application of vertical sand-gravel points (drains) to lowering of groundwater.

Commission (National Association SSSR) for soil mechanics and foundation construction KOMGF.

Scientific conference on problems of computing ground settlements during thawing.

From experience beyond the curtain:
Standards and Technical Conditions for Foundation Design in England.

If sufficient interest in having the journal translated in part or completely is shown by members of their division, financial support for the translation and publication will be sought. To express your interest write to:

Prof. J. O. Osterberg, Chairman, Exec. Comm.
Soil Mechanics & Foundations Division
Technological Institute
Northwestern University
Evanston, Illinois

NEWS FROM THE LOS ANGELES SECTION

The following news has been submitted from the Los Angeles Soil Mechanics Group by their Secretary, Mr. Irving Sherman. Five meetings were held during 1959, the Group usually meeting on the third Wednesday of January, March, May, September and November.

The speakers and subjects during 1959 were as follows:

January 21, Thomas M. Leps, "Construction of Mammoth Pool Dam"

March 18, F. O. Fricker, "The Role of Soil Mechanics in Storm Drain Construction"

May 16, Maurice Cavieux, "Demonstration of Landslide Instrumentation"

September 16, Earl B. Hall, "Pore Pressure Measuring"

November 18, Ronald F. Scott, "Arctic Soil Engineering"

The May 16 meeting was a field trip to the Pacific Palisades landslide area; the other meetings were dinner meetings in the evenings. Average attendance has been over 50 per meeting.

At the January 20, 1960 meeting, H. B. Seed will speak on "Soils Engineering in Soviet Russia."

The Group has two standing committees for purposes of furnishing advice to local agencies. One is the Committee on Cooperation of Consulting Soil Engineers with Local Building and Safety Departments; LeRoy Crandall will serve as Chairman for 1960. A second committee is now being organized under the chairmanship of R. C. Weber to review the location of seismographs proposed by the Earthquake Engineering Research Institute.

Officers of the Group for the past and coming years are as follows:

1959		1960
James R. Davis	Chairman	Vernon A. Smoots
Vernon A. Smoots	Vice-Chairman	Charles S. Howe, Jr.
Irving Sherman	Secretary-treasurer	Thomas D. Lake
Paul Baumann	Director	Frederick A. Barnes
Jimmy D. Kirkgard	Director	Lee R. Henning

APRIL NEWSLETTER

Deadline date for arrival at this office of contributions for the April Newsletter: February 25, please.

Bernard B. Gordon, Assistant Editor
Porter, Urquhart, McCreary and O'Brien
1140 Howard Street
San Francisco 3, California

Wilbur M. Haas, Assistant Editor
Michigan College of Mining and Technology
Houghton, Michigan

J. H. Schmertmann, Assistant Editor
College of Engineering
University of Florida
Gainesville, Flroida

Alfred C. Ackenheil, Editor
Civil Engineering Department
University of Pittsburgh
Pittsburgh 13, Pennsylvania